CONCRETE
AND
CONSTRUCTIONAL ENGINEERING

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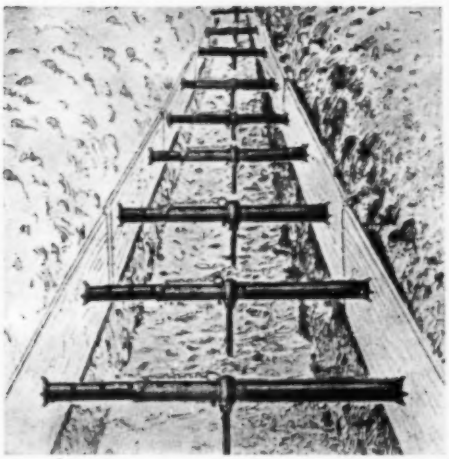

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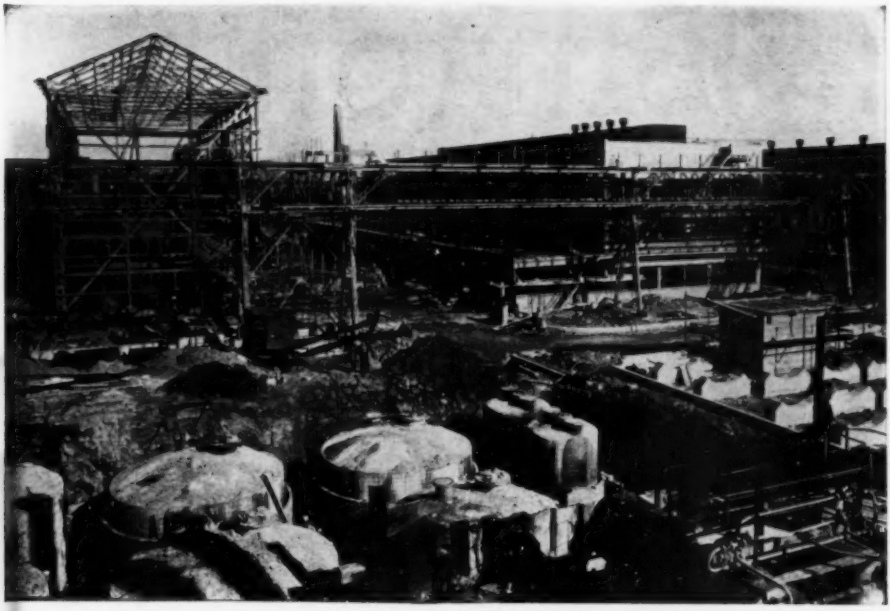

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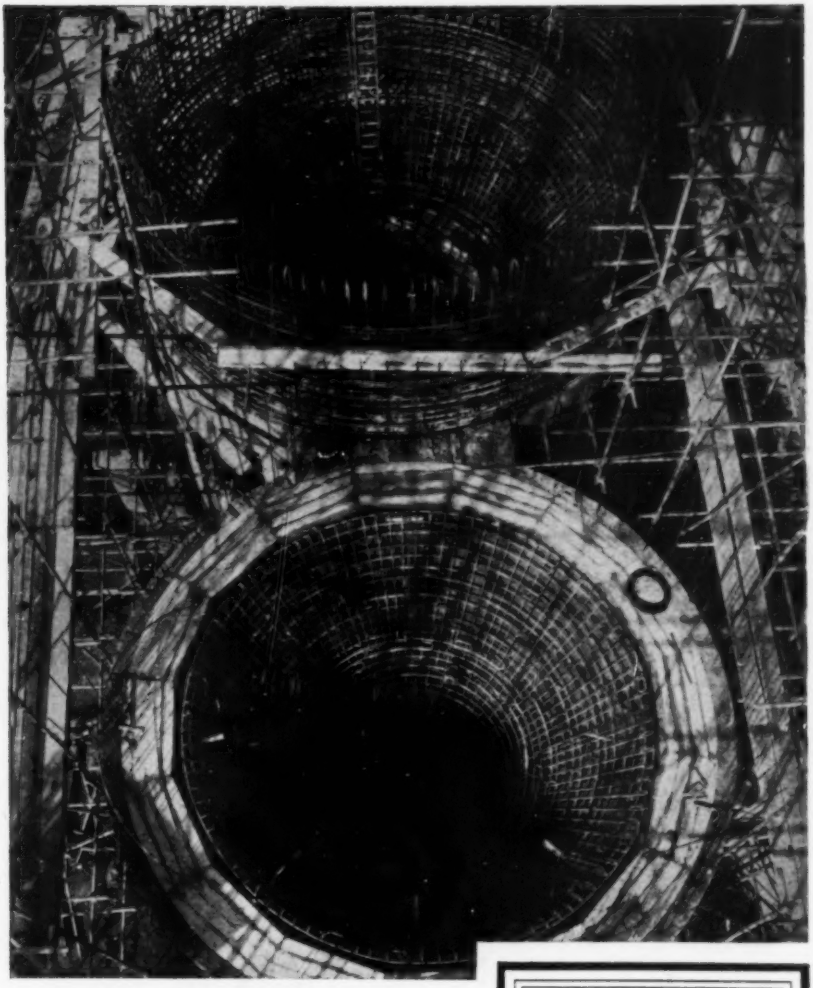

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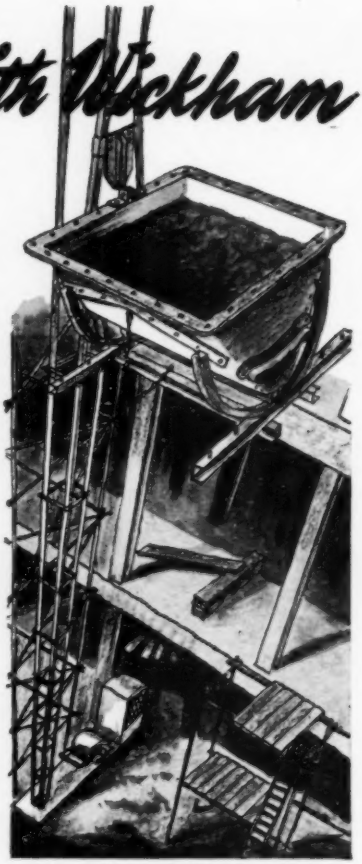

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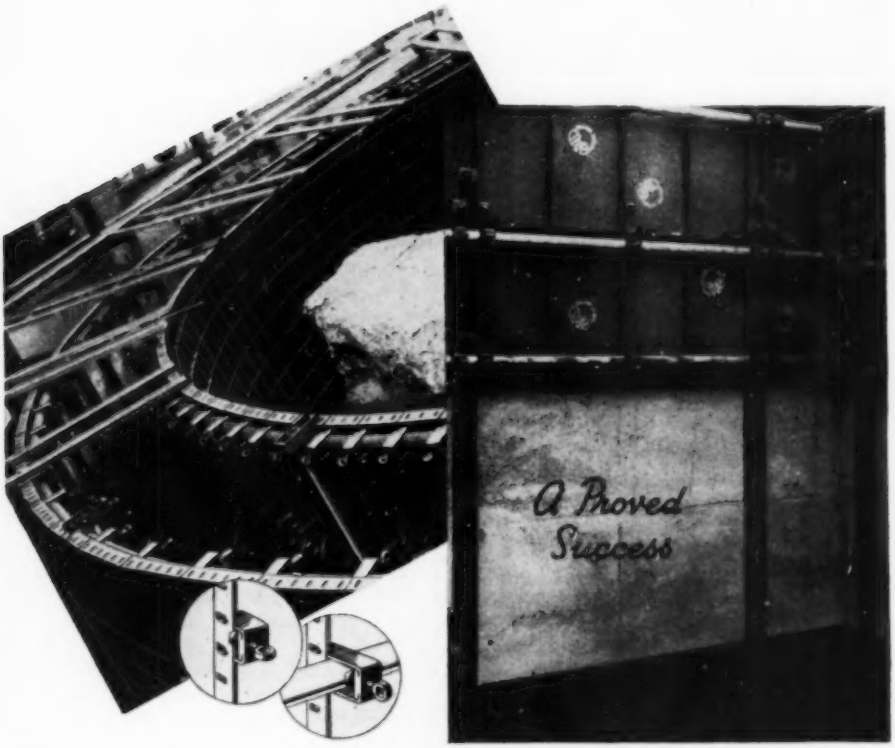

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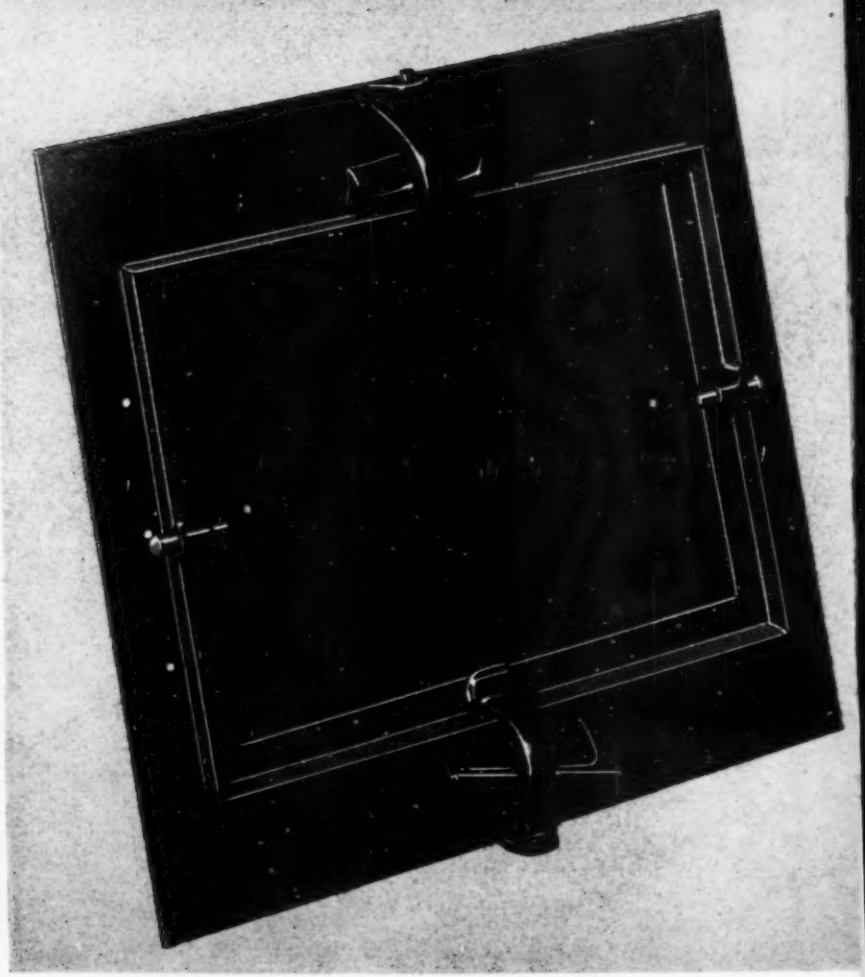

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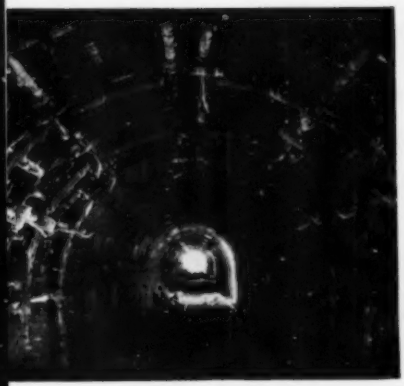

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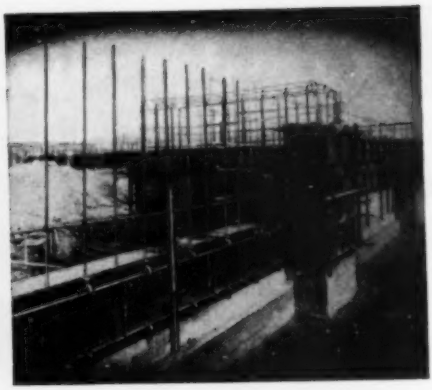

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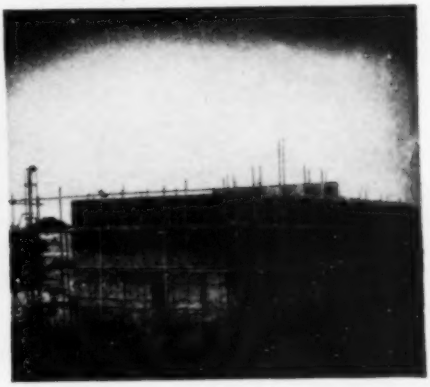

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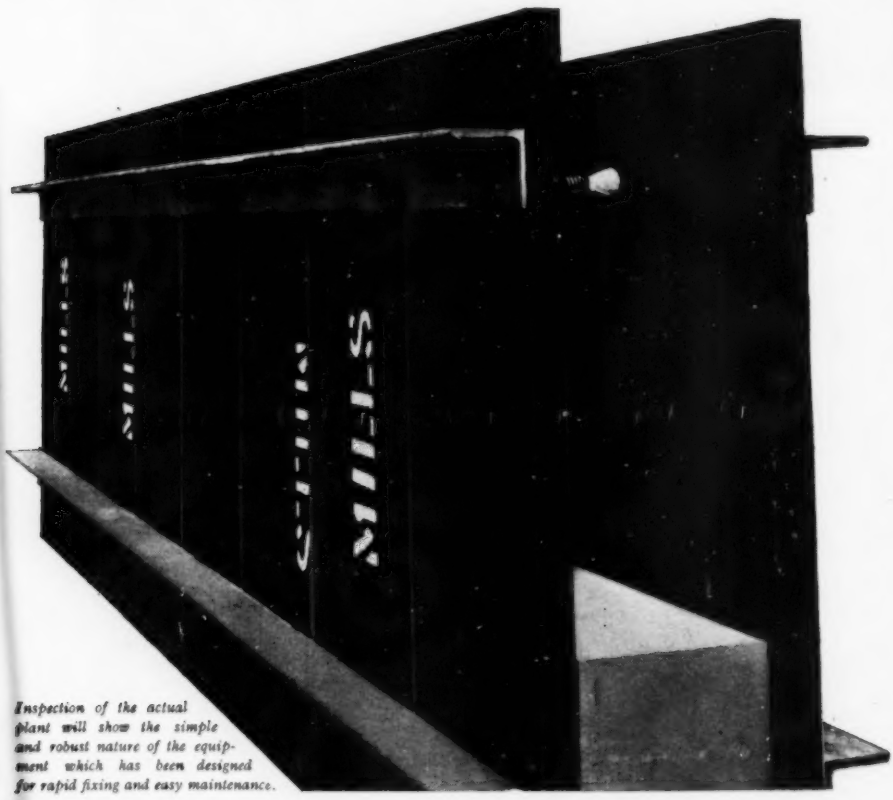

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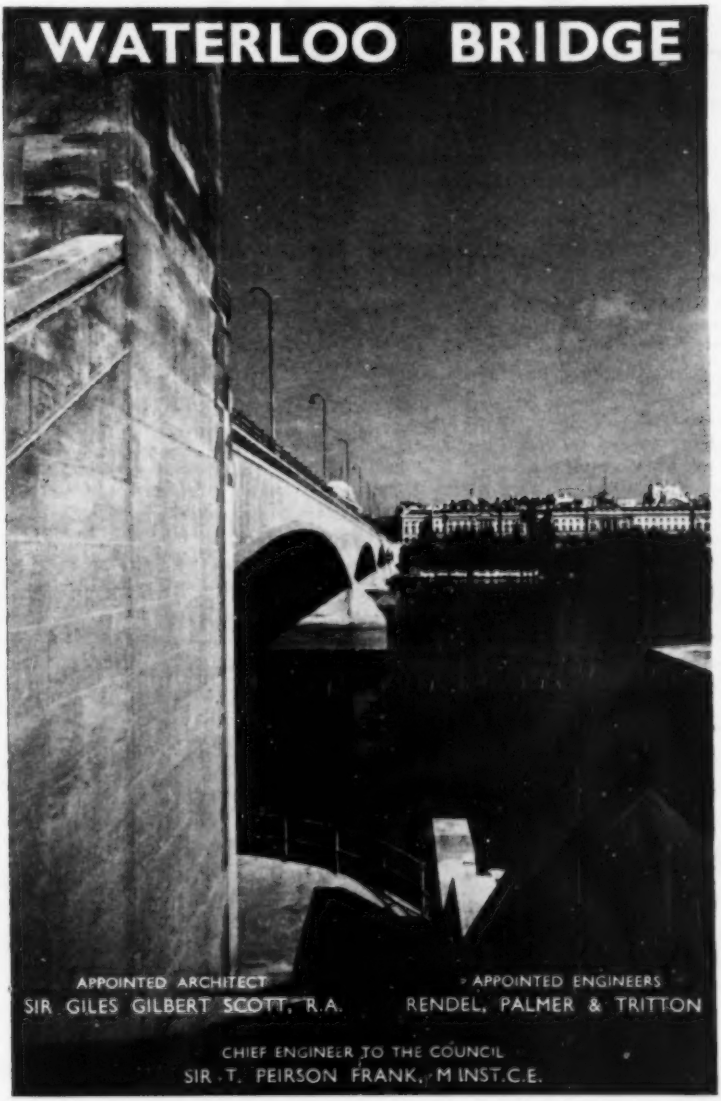

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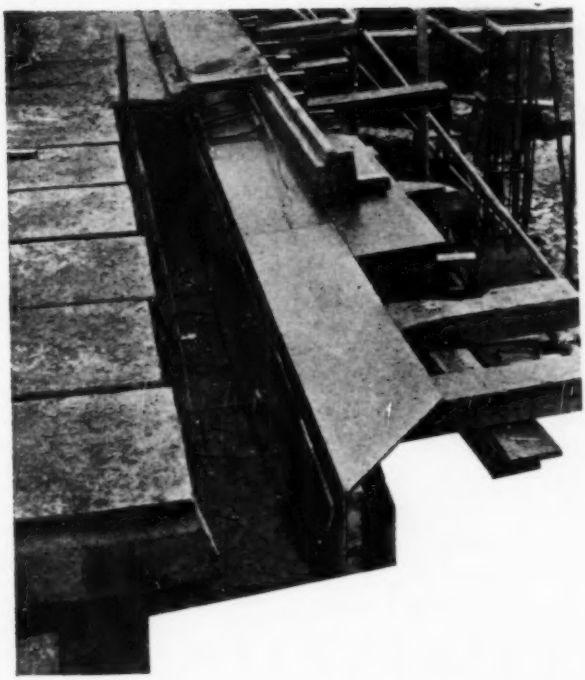

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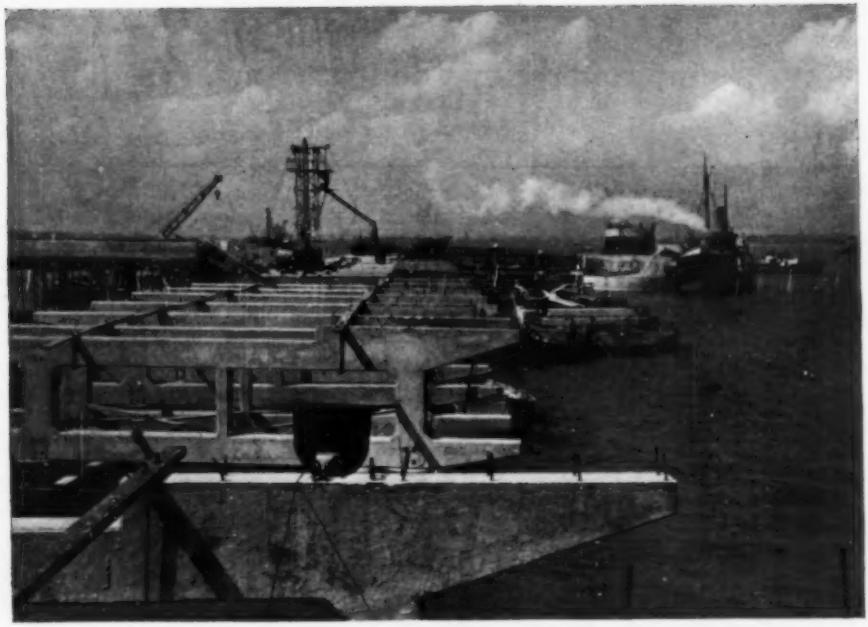

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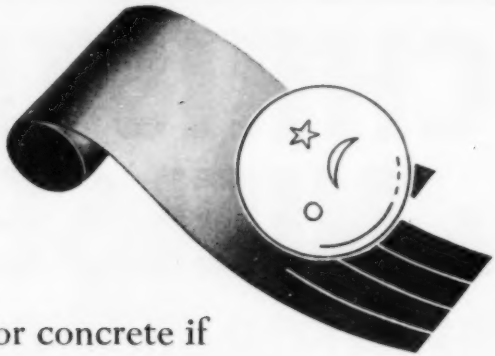

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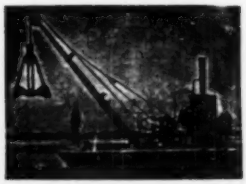

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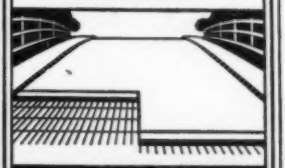

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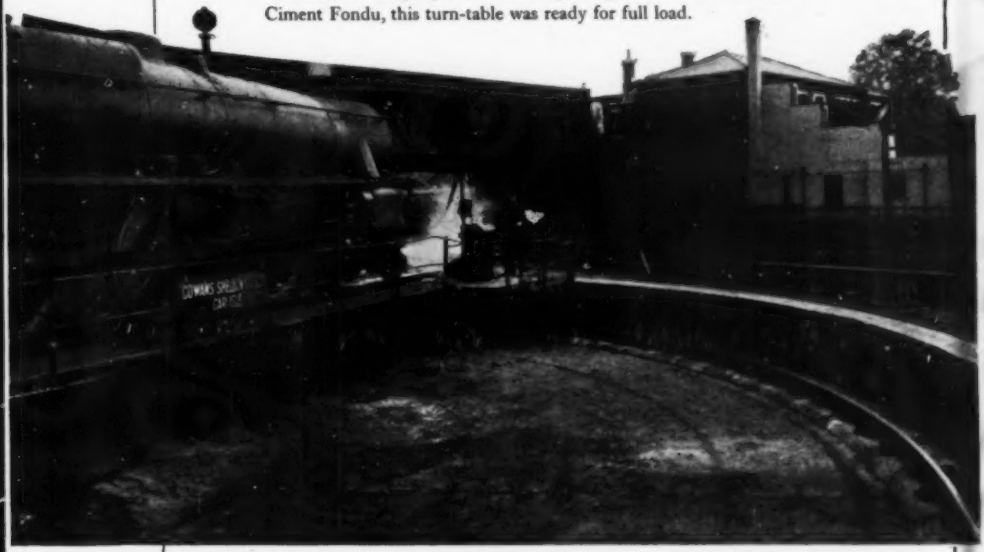

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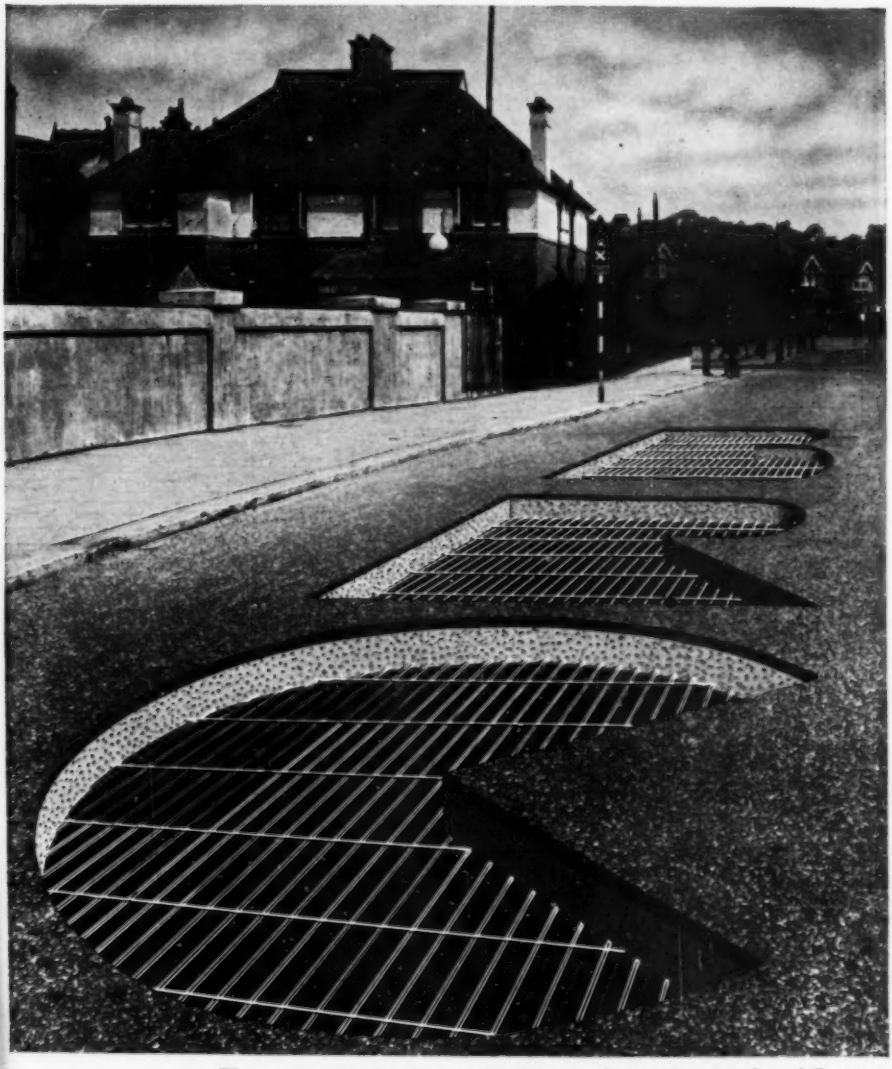

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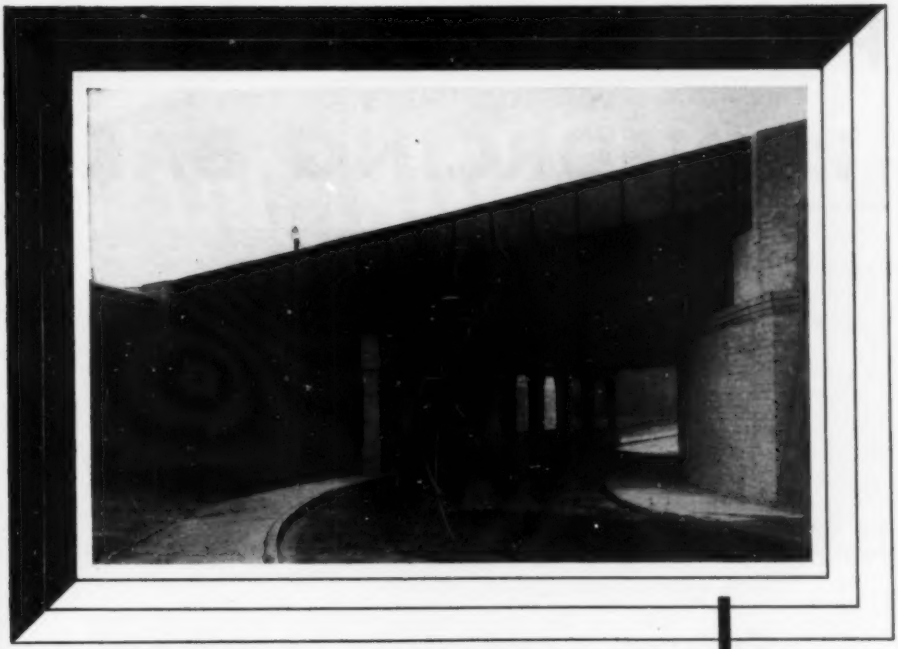

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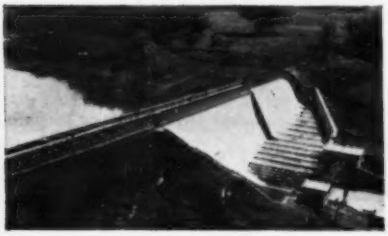

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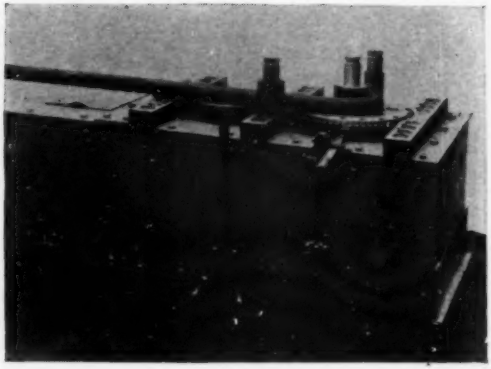

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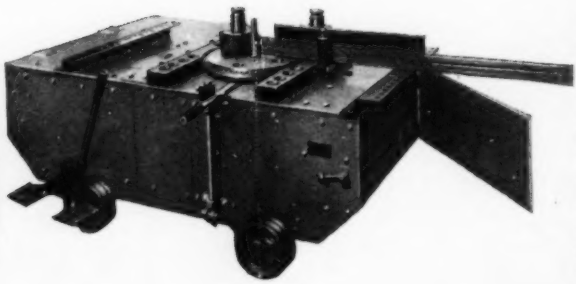

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Either of the Models illustrated can be supplied motorised or engine driven.

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This non-staining, smooth and easy working white mould oil can be used with confidence on all classes of in situ and precast concrete work where the use of a white mould oil is recommended.

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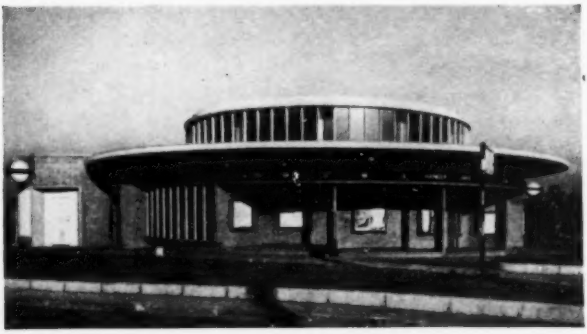

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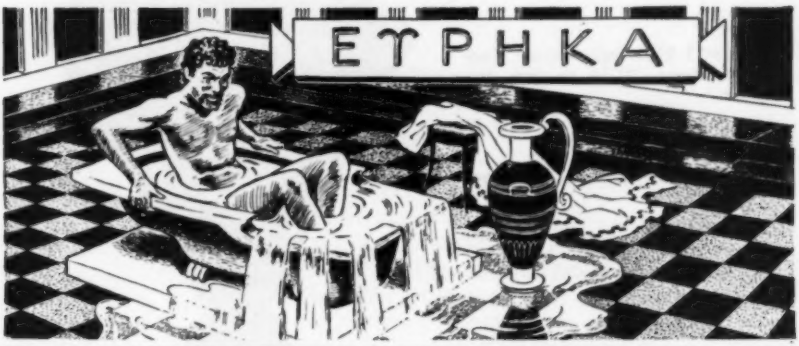
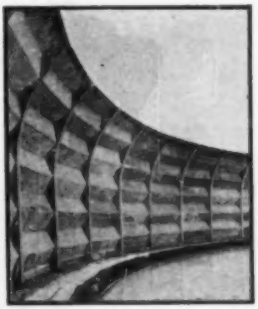


'EUREKA' cried Archimedes, as the water spilled from his bath. "If — as I see — a body removes its own bulk when immersed in liquid, here is a discovery by which I can prove lots of things ; including whether the fillet of Hiero is made of gold — or merely silver." And jumping straight from the tub he ran naked into the royal presence, eager to put the principle to the test.

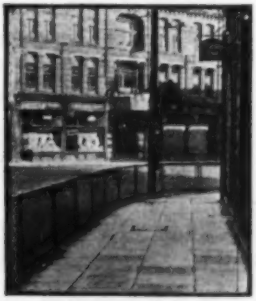
In essence, most good ideas are sound, simple, and capable of infinite application ; Expanded Metal being a typical case in point. Reinforcement for concrete and lathing for plasterwork : walkways and letter-trays : openwork partitions and loudspeaker grilles : air filters and guards for machinery — these are but a few examples of the remarkable *adaptability* of Expanded Metal, the product of one-hundred-and-one uses.



Above : 'Ribmet' used for Windshield Wall to four-and-a-half million gallon settling tank at Palliser Works, Hartlepool : designed by Indented Bar & Engineering Co. Ltd. Contractors : Costain— John Brown, Ltd.

●

Below : 'Expamet' Pedestrian Barrier for the County Borough of West Hartlepool.



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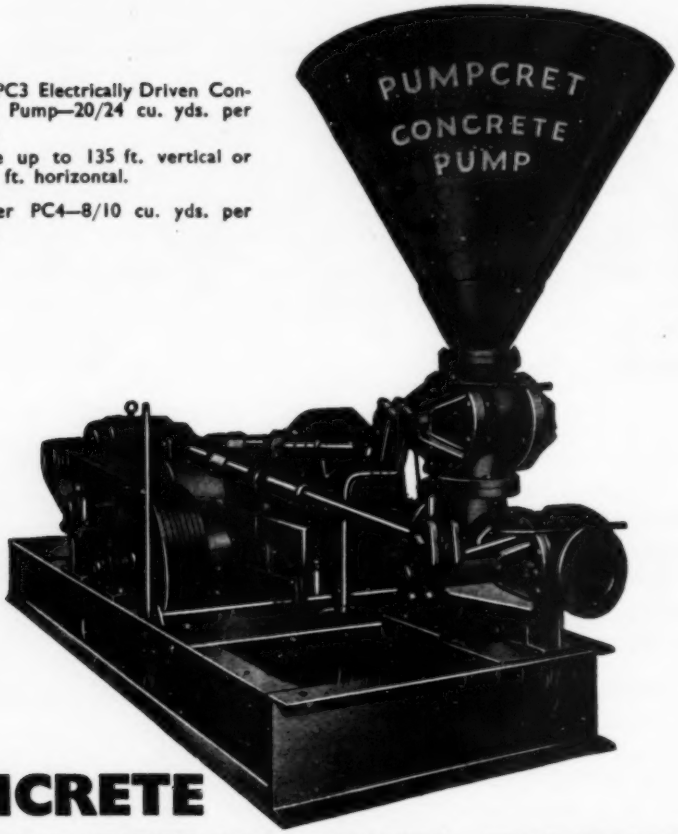

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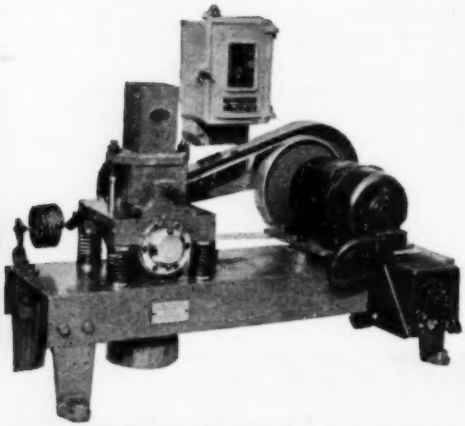

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To mark this Festival of Britain year, Acrow (Engineers) Ltd. provide these men with a unique opportunity in a competition carrying cash prizes totalling £1,000.

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As a guide to the scale and type of idea the organisers have in mind may be mentioned such examples as:—methods of speeding up bricklaying; a practical system of steel shuttering for beams and columns comparable in efficiency with the most modern methods of steel shuttering for walls and floors; reduction of cost and wastage in scaffold boarding; methods of speeding up scaffold-erection; in brief, any method of saving time, labour and/or materials in a construction-technique or, alternatively, any improvement in the design and cost of products which are incorporated in structures and become part of them.

RULES

1. The competition is open to any person resident in the United Kingdom except employees of Acrow (Engineers) Ltd. and their families. There is no entrance fee.

2. The decision of the Judging Panel will be final and legally binding and no correspondence can be entered into in connection with the competition.

3. Entries must be sent in a sealed envelope to arrive not later than July 31st,* 1951, addressed to the Managing Director, Acrow (Engineers) Ltd., South Wharf, Paddington, London, W.2, and marked "Festival of Britain Competition." No entries will be considered if received after July 31st, 1951 and no responsibility will be accepted for entries lost, damaged or delayed in the post. No entries will be returned unless accompanied by a stamped addressed envelope.

4. The prizes will be allocated as follows:—1st prize £500; 2nd prize, £200; 3rd prize, £50; 25 prizes of £5 50 prizes of £2.10.0.

5. Competitors are not limited to one entry, but not more than one prize will be awarded to any one competitor.

6. Any prize-winning idea shall become the sole property of Acrow (Engineers) Ltd. who reserve the right to adapt use or market the same in any manner they think fit.

7. In the event of the Judging Panel deciding that the quality of entries is such as not to justify the award of the whole or any part of the total prize money, the whole or any part of the same (as the case may be) not so awarded will be donated to a charitable cause associated with the Building Industry and nominated by Acrow (Engineers) Ltd.

8. Names and addresses of prize-winners will be published in this journal during September, 1951.

* *By popular request the closing date has been extended to July 31st.*

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1951

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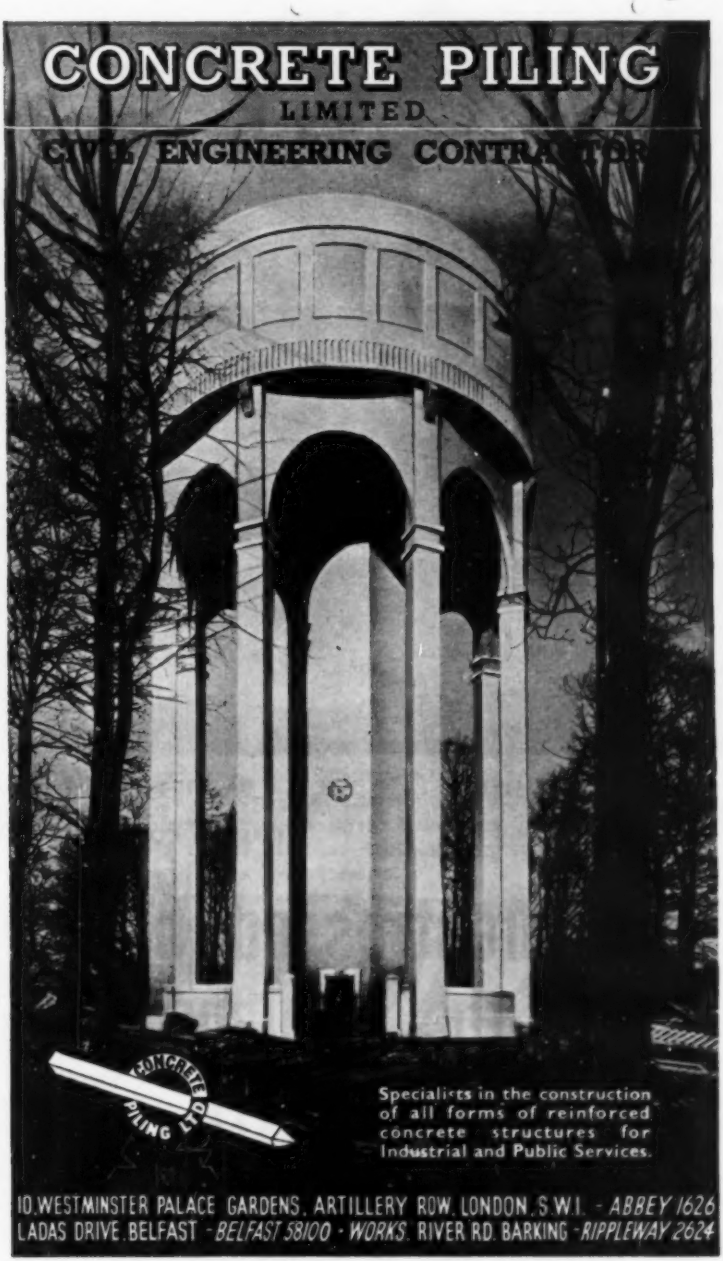
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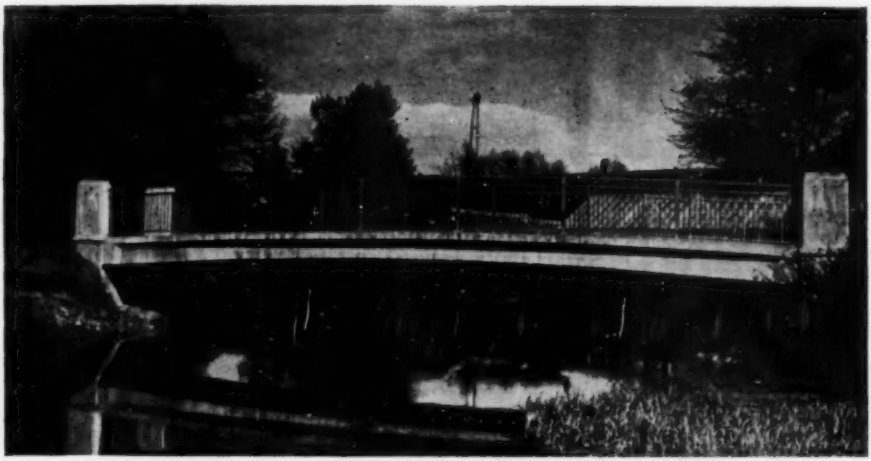

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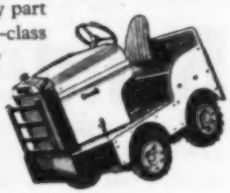

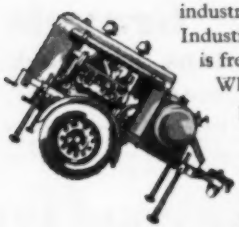

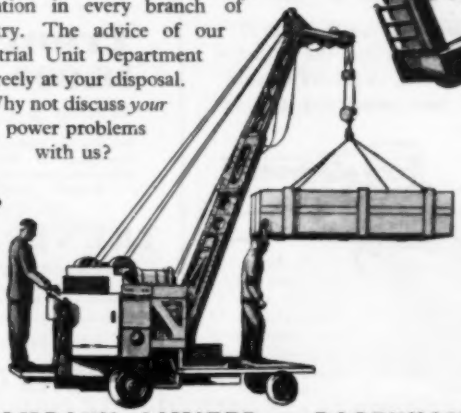

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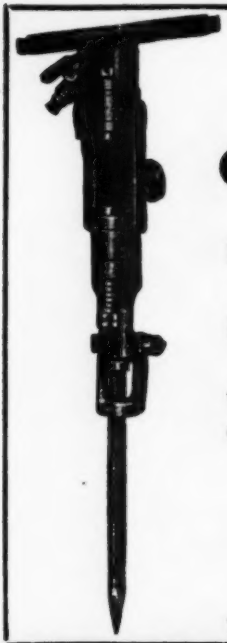

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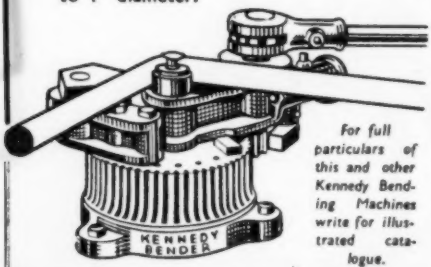
Benders

The machine illustrated is the model 1213 BAR Bending Machine for the accurate and rapid cold bending, to precise measurement, of mild steel reinforcing bars up to 1" diameter.



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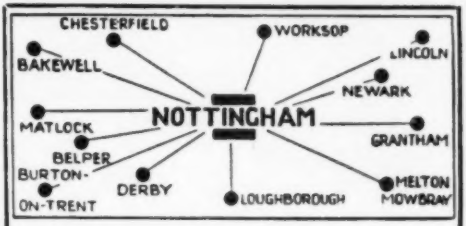

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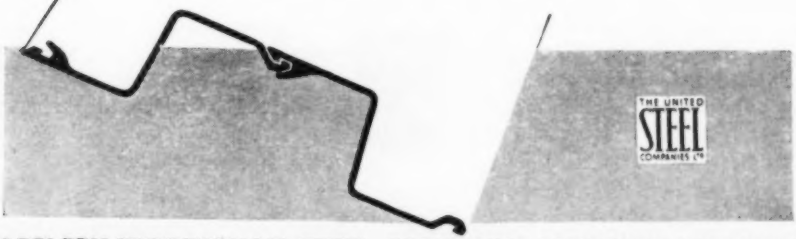

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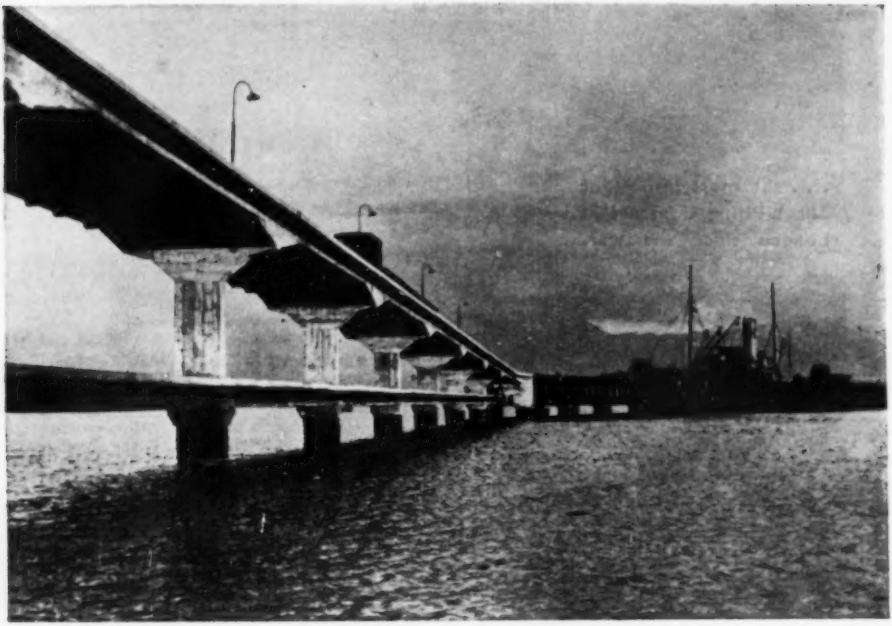

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Volume XLVI. No. 7. LONDON, JULY, 1951

EDITORIAL NOTES

Engineering and Mathematics.

As the theory of structures develops an increasing amount of higher mathematics appears to be required, so much so that it may well be doubted whether some complex mathematical processes really represent the changes of deformation and stress to which a structural member is subjected. The explanation in some cases may be that, although the basic equations which express the behaviour of the structure are not complex, the solutions of these several simultaneous equations are a complex and lengthy mathematical procedure. The art of engineering lies in the ability to interpret the results of mathematics rather than in the mathematics themselves. For example, analysis may give the bending moments which, under certain conditions, may not all be realistic. To provide resistances equal to these moments without considering the pertinence of the conditions is not good engineering. An assumption commonly made is that a member is not in a state of strain before the application of the external loads, that is, the deformations and stresses caused by piecemeal construction and by shrinkage restrained by reinforcement, shuttering, or adjacent members are commonly neglected although they may differ greatly from those caused by the load on the completed structure. The large variation of the moment of inertia due to changes of shape, the amount of reinforcement, and the amount of cracking of the concrete is also generally ignored, and conditions of free support are often assumed when such is not the case. Such factors must be taken into account in design by considering whether the results of the mathematical analysis must be increased at one point, or may be relaxed at another. This view is expressed in a publication * of the Road Research Laboratory in which about thirty methods of designing roads and runways are reviewed. The methods include empirical ones based on the classification of the soil, methods based on the strength of the soil, methods based partly on theory and partly on experience, and others which are wholly theoretical. There is no method that is generally accepted, and it is shown that many have little in common either as regards the assumptions or the results. The most reliable methods give only a rough idea of the thickness of the slab required. The best empirical methods give more reliable results than methods which are wholly theoretical, but, as is stated in the publication, all methods require a considerable amount of common sense and experience on the part of the engineer

* "Pavement Design for Roads and Airfields." Road Research Technical Paper No. 20. (London: H.M. Stationery Office. 1951. Price 1s. 9d.)

who uses them. The modification made recently to the formulæ of Professor H. M. Westergaard is considered to be the most reliable method for calculating the stresses due to traffic on concrete roads, and another theory is recommended for calculating the stresses due to temperature changes.

As was the case with nearly all engineering structures up to a century ago, the design of concrete roads has been mostly based on experience, judgment, or guesswork. It is only to be expected that in the course of time attempts would be made to express mathematically the behaviour of a concrete road, as has been done in the case of other structures. In view of the many influences of unknown magnitude that affect the stresses, it is not remarkable that a satisfactory analysis is difficult to devise. Because of the great loads to which roads and runways are now subjected, a reliable method of design is more than ever necessary if failures or waste are to be avoided. Experience allied to fundamental and applied research is the only source from which such a method can be evolved. The publication suggests that the beginnings of such a method may be seen in the analysis of elastic systems of members composed of layers. This analysis is highly mathematical, and is not regarded as suitable at present for practical design, but may eventually be made so if modifications based on research and experience are introduced to take into account the non-elastic behaviour of the materials.

The emphasis on mathematics in much structural design may be traced to the manner in which engineers are trained, and to the common use of codes and by-laws. Whereas an experienced engineer may often be able to satisfy himself that a design is sound without much calculation, it is a different matter if he has to justify his opinion to someone who is more likely to accept the result of a calculation rather than an opinion unsupported by calculation. In this respect it must not be overlooked that the implication of conventional calculations is that if the results obtained from a certain method of calculation are within certain limits the design is sound. In reinforced concrete it cannot be accepted that calculated stresses, for example, are the actual stresses, although the usual methods of calculation lead to sound structures.

Most engineering subjects have during the past hundred years or so become increasingly considered as branches of applied mathematics, and only students with an aptitude for mathematics are likely to pass the examinations necessary for qualification, while those who may have sound engineering instinct but little taste for mathematics may fail to do so. The system by which a young man was indentured to a practising engineer, and which has now been replaced almost entirely by a more academic training, gave those without, or with, mathematical propensity an excellent opportunity to gain engineering experience. More knowledge of practical design is likely to be gained by working with experienced engineers than from books or lectures. Applied research, not only in the design of concrete roads but in many other branches of structural engineering, may provide the opening for the engineer with little feeling for mathematics, because it is from realistic research that deductions of engineering value are obtained, and these can be obtained only by those who realise the practical implications. In recent years there has been in civil engineering a tendency to concentrate on research in the laboratory rather than on the site, and this is no doubt one of the reasons why so few new ideas in civil engineering and building construction have originated in Great Britain during the past thirty years.

Concrete Structures at the Festival of Britain.

SOUTH BANK, LONDON, EXHIBITION.

CONCRETE structures of uncommon design at the South Bank (London) Exhibition of the Festival of Britain described in the following include the substructure of the Dome of Discovery (*Fig.* 1), an entrance canopy (*Fig.* 4), a restaurant roof (*Fig.* 5) and a footbridge (*Fig.* 7) of prestressed concrete, and the Natural Resources Pavilion (*Fig.* 12). The superintending consulting civil engineers are Messrs. Freeman, Fox & Partners, who also designed the reinforced concrete work

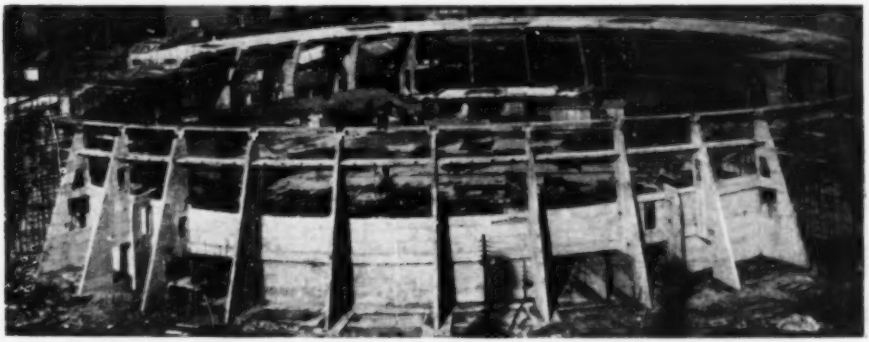


Fig. 1.—Galleries in Course of Construction under the Dome of Discovery.

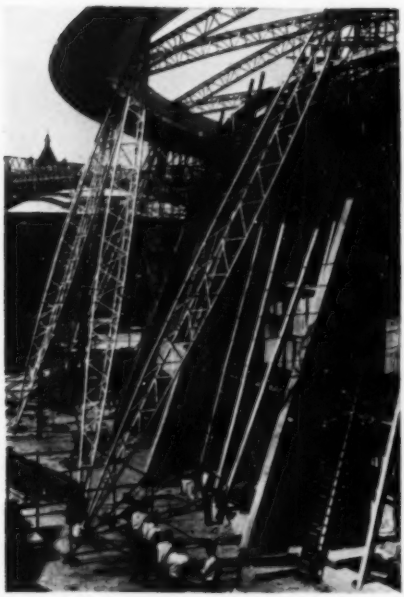


Fig. 2.—Support of Dome and Wall.

for the Dome of Discovery, the architect for which is Mr. Ralph Tubbs. The consulting engineers for the other structures described are Messrs. Ove Arup & Partners. The architects for the entrance canopy, restaurant roof, and Natural Resources Pavilion are the Architects Co-operative Partnership, and for the footbridge Messrs. Fry, Drew & Partners. Messrs. Richard Costain, Ltd., were the contractors for all the structures except the footbridge, which was constructed by Messrs. Kirk & Kirk, Ltd.

Substructure of the Dome of Discovery.

The substructure of the Dome of Discovery comprises galleries at four levels and a circular drum wall. The aluminium roof is carried on tubular steel struts (*Fig.* 2) which also support the outer edge of the apron and resist the wind forces on the roof without imposing restraint on the substructure. The galleries are supported on separate foundations and are designed for a superimposed load of 100 lb. per square foot. The floors are generally of precast hollow reinforced concrete beams, which were used to save

shuttering and time. The beams are up to 21 ft. in length and cantilever 5 ft. in one of the intermediate galleries. Because of the irregular curved shapes of the floors, the edges are of cast-in-situ reinforced concrete. Three of the galleries are supported on steel beams and columns, but the highest gallery, which is 35 ft. above the ground floor, is carried on steel beams supported on radial reinforced concrete fins 15 in. thick and reinforced vertically with ¾-in. bars at 7 in. and 8 in. centres (*Fig.* 3).

The circular drum wall, the height of which varies from 9 ft. to about 38 ft., has an internal diameter of 260 ft., the centre of the circle being 29 ft. eccentric in relation to the centre of the dome. The wall is of double construction. The inner wall is vertical and is generally of breeze blocks, except at the south end where it is of reinforced concrete 9 in. thick (*Fig.* 1). The outer wall is inclined and is built of steel mesh rendered with mortar and supported on precast reinforced concrete rakers. Between the walls are triangular reinforced concrete counterforts which are arranged radially and support the framing of the outer wall and the lower ends of the aluminium girders supporting the apron below the dome. The counterforts are 12 in. and 9 in. thick, depending on the height, and are connected at the top by a 6-in. reinforced concrete slab. The 6-in. slab supports the tops of the precast rakers which span between the slab and a ground-beam. The rakers are 8 in. by 6 in. and are 33 ft. 2 in. long. Each weighs about ¾ ton and was erected by hand-operated tackle (*Fig.* 2). Other reinforced concrete slabs are provided at lower levels to support the ventilation equipment.

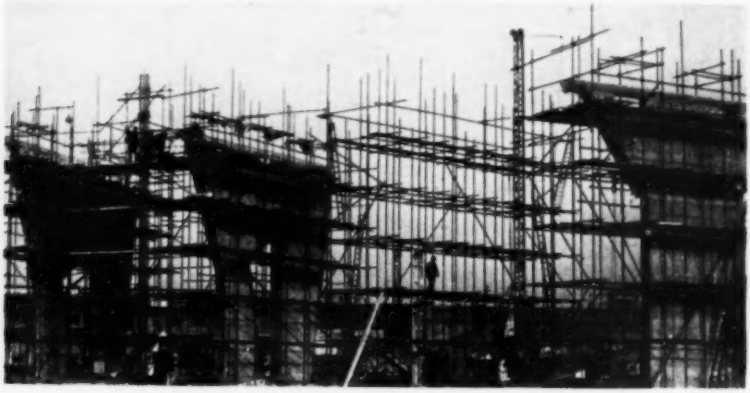


Fig. 3.—Columns under the Dome of Discovery.

Fig. 4.—Double-cantilever Canopy at Entrance.

Entrance Canopy.

One of the entrance canopies (*Fig.* 4) is a double-cantilever reinforced concrete structure 53 ft. 9 in. wide and 68 ft. 9 in.

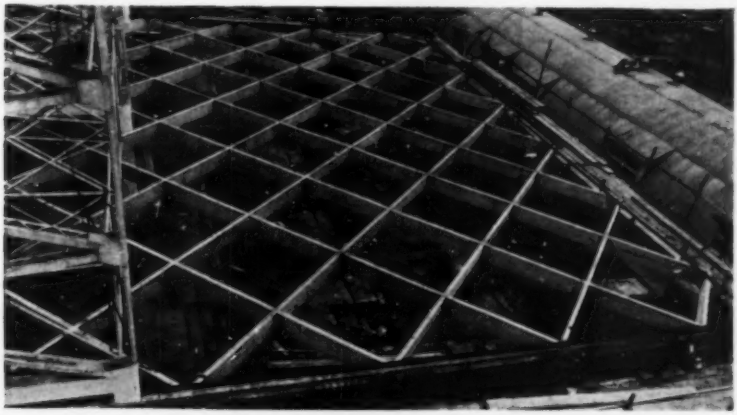


Fig. 5.—Prestressed Concrete in Roof of Restaurant during Construction.

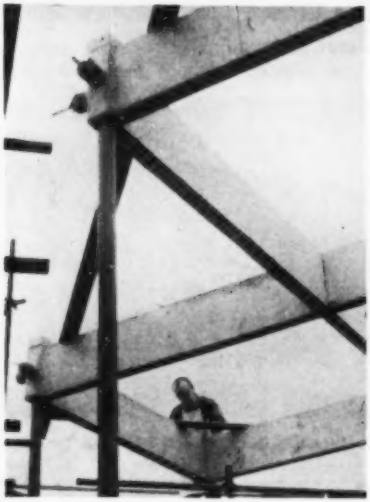


Fig. 6.—Columns and Ribs in Roof of Restaurant.

long, and comprises principally two thin slabs inclined in opposite directions at an angle of 15 deg. to the horizontal. The slabs are supported at 10 ft. 6 in. centres on six double-cantilever ribs 7 in. thick. The ribs, the greatest overhang of which is about 25 ft., are above the slabs and are supported on a central valley beam, the average depth of which is 2 ft. 9 in. and the width 3 ft. Because of the unsymmetrical arrangement of the slabs in plan in relation to the beam, the two arms of each rib are unequal in length. The difference in the bending moments at the support is resisted by the horizontal torsional action of the beam which is reinforced by continuous inclined binding to resist the torsional stresses. The unbalanced twisting moments on the beam are resisted by two columns which support the beam at points 27 ft. 6 in. apart and are of dumb-bell cross-section. Resistance to the bending moment to which the columns are consequently subjected is provided by arranging the columns eccentrically on their bases so that the pressures due to the dead loads are distributed uniformly on the ground.

Restaurant Roof.

The roof of the restaurant adjacent to York Road is a novel application of prestressed concrete (*Fig.* 5). The roof, which is about 50 ft. by 60 ft., comprises a diagonal grid of concrete ribs supported on a wall along one side and on 3-in. diameter steel columns at 10 ft. centres along three sides, along two of which the columns are set in 5 ft. from the edge of the roof, thereby causing the ribs to cantilever beyond the columns.

The ribs are $3\frac{1}{2}$ in. thick and 15 in. deep, and were precast in lengths of about 6 ft. 9 in. Two $\frac{5}{16}$-in. mild steel bars in the top and bottom and $\frac{3}{16}$-in. links

at 9 in. centres were provided to resist stresses during erection. A 1¼-in. diameter hole was formed longitudinally through each rib at different levels to receive the prestressing cable, the position of which varies to conform to the negative and positive bending moments in each rib due to the total dead load, the wind, and an imposed load of 15 lb. per square foot. The calculation of the bending moments, without taking into account the effect of prestressing, is complex since the interaction of the intersecting ribs produces a structure that is statically indeterminate to at least the tenth degree. The calculation was made by an analysis based on the Müller-Breslau theory. The cables are arranged so that the compressive stresses produced by prestressing offset the tensile stresses due to the total dead and live loads.

After erection of the ribs on falsework at the level of the roof, 4-in. square concrete blocks were cast in situ at each intersection. When the concrete had hardened, the cables threaded through the holes were stretched and anchored by the Freyssinet method. *Fig.* 6 shows the anchorage cones at the ends of the ribs. Since only one rib could be prestressed at a time, the loads from intersecting ribs would not be acting when the first ribs were prestressed, so that a sequence of prestressing had to be specified, and kentledge, in the form of weights up to 15 cwt., had to be applied temporarily at the intersections of the ribs to prevent upward deflection of the ribs during prestressing. After prestressing, the kentledge was removed and the rise of the centre of the roof was ⅜ in., but this was eliminated when the roof covering was complete.

Fig. 7.—Prestressed Concrete Footbridge.

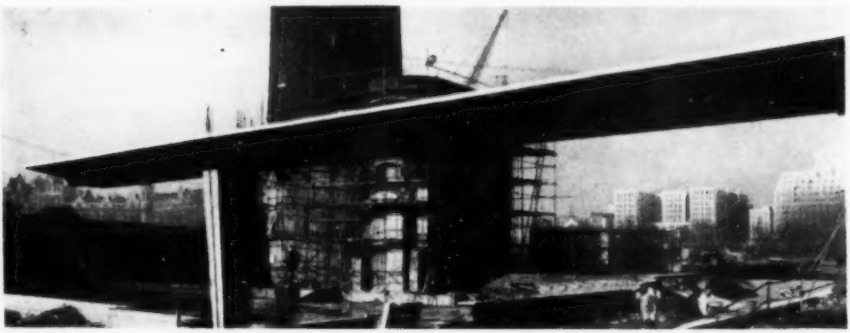


Fig. 8.—Prestressed Concrete Footbridge During Construction.

Footbridge.

The footbridge (*Figs.* 7 to 11) from the Waterloo bridge entrance to the forecourt of the Royal Festival Hall is a prestressed concrete continuous-girder nearly 300 ft. long, and is designed for an imposed load of 100 lb. per square foot. The structure comprises five columns and a tee-girder,

22 in. deep, of four spans of about 76 ft., 59 ft., 76 ft., and 54 ft. and a cantilever of 19 ft. The ratio of depth to span is about 1 to 40. Mild steel reinforcement is provided transversely in the top of the tee, which forms a deck 11 ft. 10½ in. wide. Twenty-four cables in the three longer spans and fourteen cables in the shortest span provide the longitudinal prestress. Some of the ducts for the cables were formed by inflated rubber tubes and others by permanent steel sheaths enclosing the cables. The level of the cables varies according to the positive and negative bending moments in the girder, and the stretching force was such that theoretically the maximum compressive stresses do not exceed 2000 lb. per square inch and tensile stresses are not developed. The mixture of the concrete is nominally 1 : 2 : 4 with specially graded aggregates, a low water-cement ratio, and compaction by immersion vibrators. The minimum strengths of cubes were 4000 lb. per square inch at seven days and 6000 lb. per square inch at 28 days.

The deck was prestressed in six main operations (*Fig.* 11) depending on the availability of the site and the desirability of limiting the length of waved cable to be stretched at each operation in order to reduce frictional losses. (1) One-and-a-half spans over columns 36 and 37 were constructed in situ (*Fig.* 8) and eighteen of the twenty-four cables were stretched from both ends by jacks at column 36 and at a temporary gap at the middle of the span between columns 37 and 38; a precast concrete block (similar to that in *Fig.* 9) to which the ends of the cables are anchored is provided at the gap. (2) Two cables looped in the short overhang at column 36 were stretched by jacks at the gap. (3) Eighteen of the twenty-four cables in one-and-a-half spans

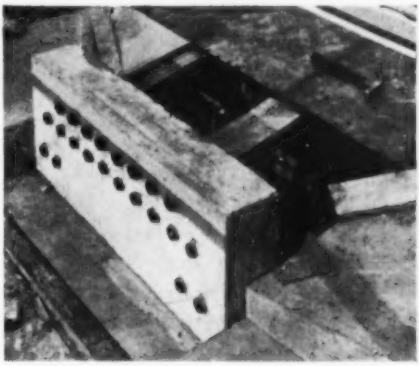
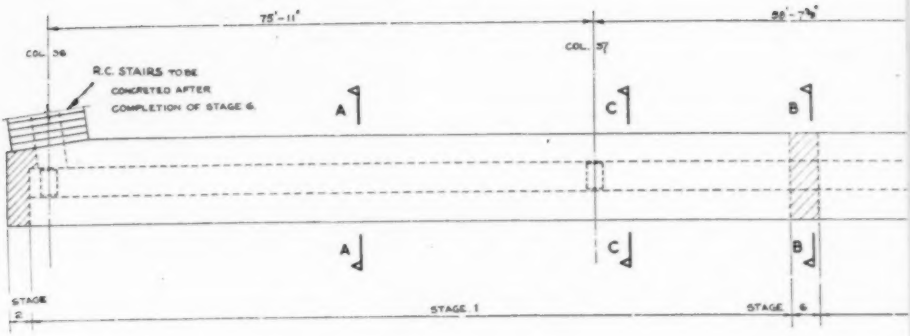


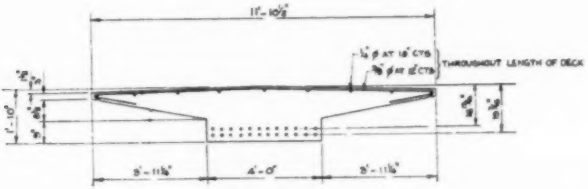
Fig. 9.—Precast Concrete Anchor Block.

over columns 38 and 39 were stretched by jacks at the gap and at column 39; also, ten of the fourteen cables in the span between columns 39 and 40 were stretched. (4) Two cables looped in the 19-ft. cantilever beyond column 40 were stretched and anchored at column 39. (5) Two pairs of cables looped over column 39 were stretched by jacks at the end of the cantilever for one pair and at the temporary gap for the other pair. (6) Four long cables extending from columns 36 to 39 were then stretched.

Columns 36, 38, and 40 were constructed with temporary hinges at the top and bottom (*Fig.* 10) to allow free movement of the deck during prestressing. The recesses at the hinges were packed with dry mortar when the prestressing of the deck was finished. All the columns were prestressed by a cable on each face, the cable lying exposed in a groove. The cables are embedded in the bases (*Fig.* 11) and anchored in the top of the deck. The columns are reinforced with sufficient mild steel bars to

Fig. 10.—Temporary Hinge in Column.





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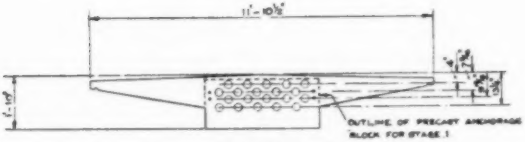

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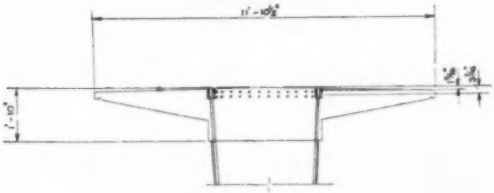
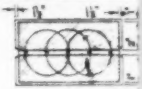


SECTION C-C.

PLAN OF DECK SHOWING STAGES OF CO



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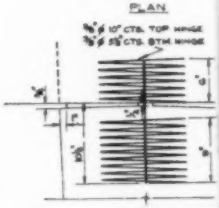

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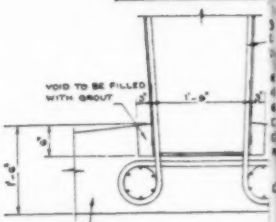

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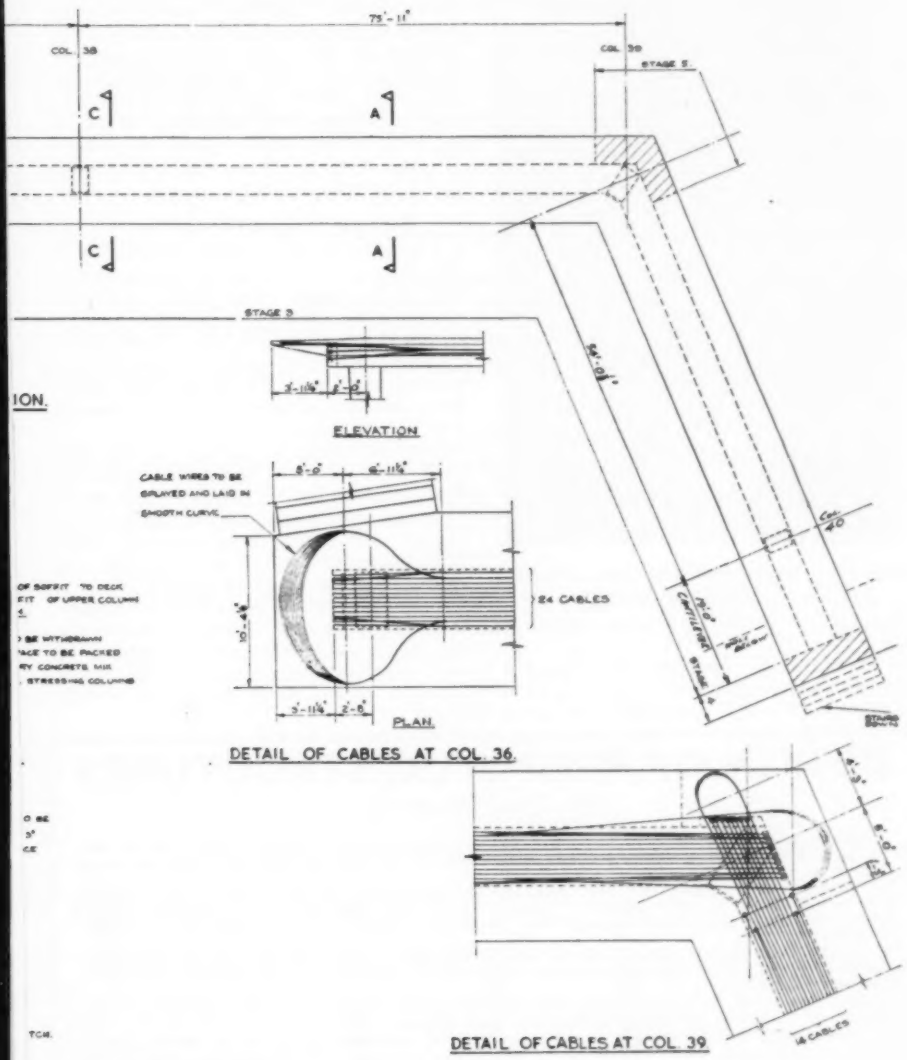
Fig. 11.—Details of Pre



Concrete Footbridge.

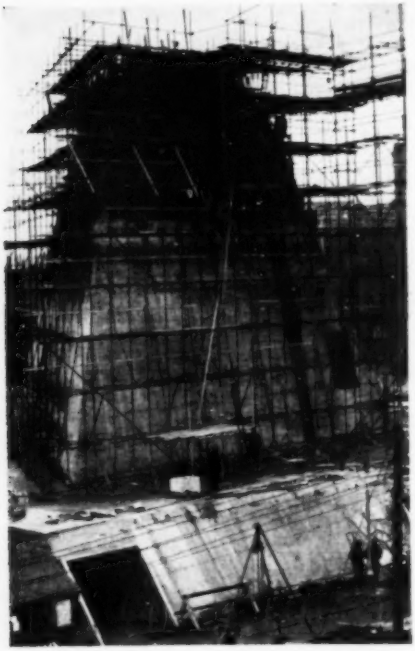


Fig. 12.—Natural Resources Pavilion During Construction.

stabilise the structure during construction.

The prestress indicated on pressure-gauges was compared with the extensions of the cables, and generally the actual extensions agreed fairly well with those calculated. Two of the four long cables in operation (6) were left exposed when the 4-ft. temporary gap was concreted. Extensometer readings taken during stretching to ascertain the reduction of prestressing force due to friction showed that the stress at the middle of the cables was about half of that at the anchorage; this condition was acceptable since it applied to only four out of 24 cables. The prestressing was in accordance with the Freyssinet system.

Natural Resources Pavilion.

The reinforced concrete structure for the Natural Resources exhibit comprises two main parts. The lower part, or plinth, which is irregular in plan, is 9 ft. high and has sloping solid concrete walls, covered with turf, supporting a hollow-tile slab of spans up to 30 ft. The upper part, shown in course of construction in *Fig.* 12, is about 70 ft. high and is a hollow tetrahedron having a base-line 73 ft. long. The walls are thin reinforced concrete slabs cast in situ and later faced with precast slabs having projections shaped and coloured to imitate coal. Within the tetrahedron is a staircase, the three lower landings of which are triangular flat slabs supported on circular columns. The two upper landings are also triangular in plan and are carried on the inwardly-sloping walls. The stairs between the landings are clear of the walls.

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By B. ERIKSEN, A.M.I.Struct.E.

Example of a Continuous-Girder Bridge with Symmetrical Girders.

The application of the method described in the first part of this article will be explained by the design of the continuous-girder bridge in *Fig.* 4, the girders of which are at 6 ft. centres. Structure joints are provided at A and A′ so that the three spans between the joints form a separate structural member. The depth of the girders is 4 ft. at the middle of each span and 8 ft. 9 in. at the piers, the rate of increase in depth being such that $2r = 2$. By calculation from the dimensions given, the moments of inertia at the middle (I_C) and at the supports (I_A) are 13·6 ft.⁴ and 122 ft.⁴ respectively. Therefore $n = \frac{13{\cdot}6}{122} = 0{\cdot}11$. The depth of the girder at intermediate points should be calculated from (1) as the girders are tee-beams, but formula (2), which strictly applies only to rectangular beams, can be used with sufficient accuracy for practical purposes. From (2) the depth d_x (in inches) at any section is $\frac{48}{\sqrt[3]{n_x}}$, where

$$n_x = 1 - (1 - 0{\cdot}11)\phi^2 = 1 - 0{\cdot}89\phi^2.$$

By substitution the depths are 48·9 in. at $\phi = 0{\cdot}25$, 52·2 in. at $\phi = 0{\cdot}50$, 60·5 in. at $\phi = 0{\cdot}75$, and 70·3 in. at $\phi = 0{\cdot}875$. The depths at $\phi = 0{\cdot}25$ and $\phi = 0{\cdot}50$ are adopted, but those at $\phi = 0{\cdot}75$ and $\phi = 0{\cdot}875$ have been adjusted to give a smooth curve.

Slopes and Fixed Points.—The length of each end span is 76 ft. and, by substitution in formulæ (3) and (4), $EI_C\alpha_1$ is 16·32 feet units and $EI_C\beta_1$ is 10·41 feet units. Therefore $EI_C(\alpha_1 + \beta_1)$ is 26·73 feet units. Similarly for the middle span, the length of which is 81 ft., $EI_C\alpha_2$ is 17·39 feet units, $EI_C\beta_2$ is 11·10 feet units, and therefore $EI_C(\alpha_2 + \beta_2)$ is 28·49 feet units.

Commencing at the left-hand support of the first span, where the girder is freely supported, the fixed-point distance is zero, and the slope-term in feet-units at the right-hand support of the first span due to $M = 1$ acting there is

$$EI_C\gamma_B = EI_C\alpha_1 = 16{\cdot}32.$$

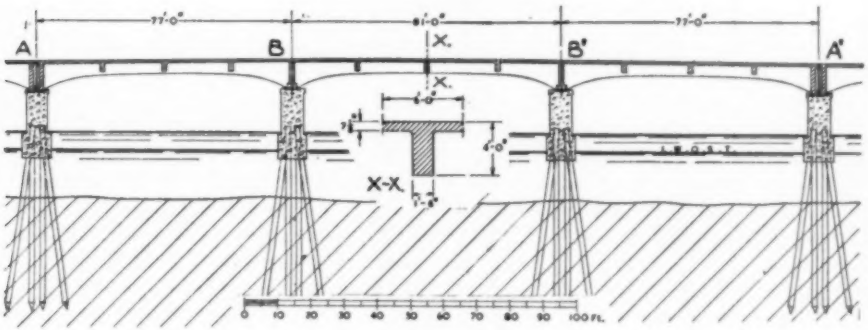


Fig. 4.

* Concluded from June, 1951.

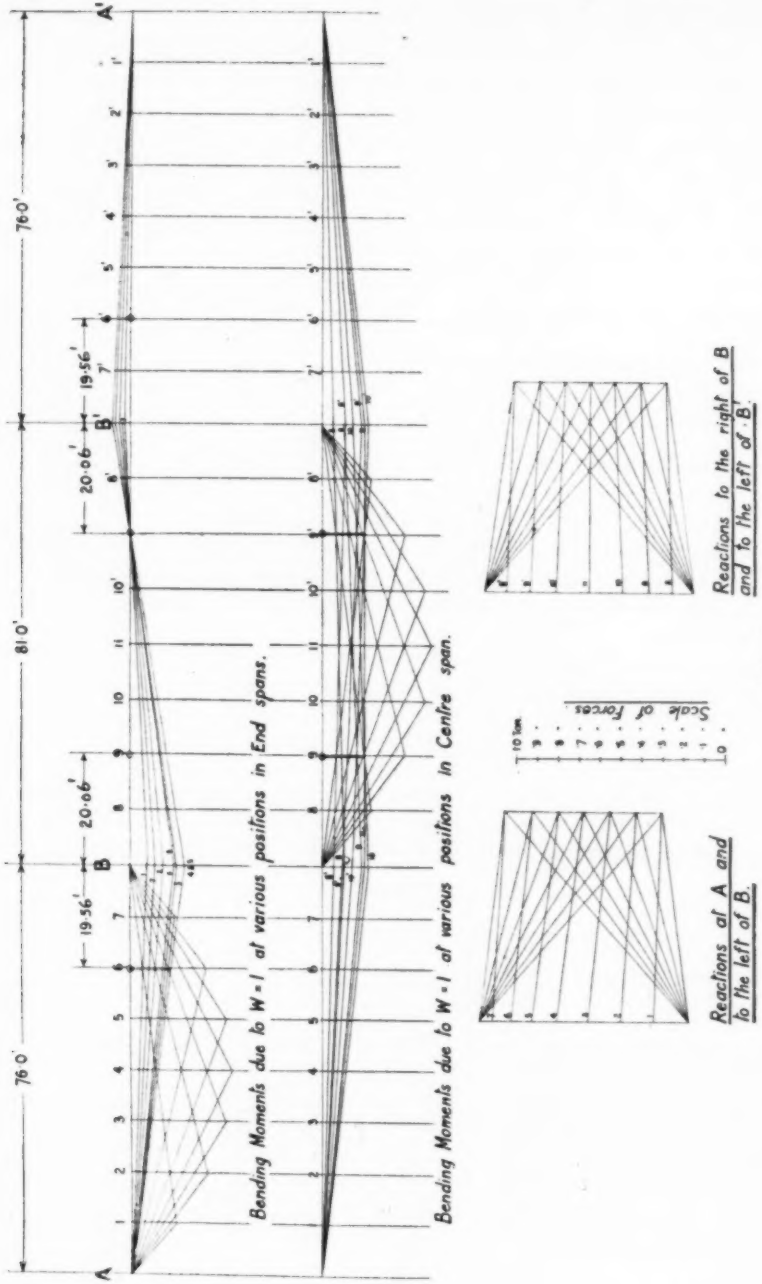

A
A'
B
B'
76·0'
81·0'
19·56'
20·66'
Bending Moments due to W = 1 at various positions in End spans.
Bending Moments due to W = 1 at various positions in Centre span.
Reactions of A and to the left of B.
Reactions to the right of B and to the left of B'.
Scale of Forces
10 Tons

Fig. 5.

The distance of the left-hand fixed point in the middle span is, from the equations established in the article in this journal for January, 1948,

$$a_2 = \frac{\beta_2 l}{\alpha_2 + \beta_2 + \varepsilon_2} = \frac{11{\cdot}10 \times 81}{28{\cdot}49 + 16{\cdot}32} = 20{\cdot}06 \text{ ft.}$$

The slope-term at the right-hand end of the middle span, due to $M = 1$ acting there, is

$$EI_C\gamma_{B1} = \alpha_2 - \beta_2 \frac{a_2}{l_2 - a_2} = 17{\cdot}39 - \frac{11{\cdot}10 \times 20{\cdot}06}{81 - 20{\cdot}06} = 13{\cdot}7 \text{ ft.}$$

The distance of the left-hand fixed point in the third span (freely supported at the right-hand support) is

$$a_3 = \frac{10{\cdot}41 \times 76}{26{\cdot}73 + 13{\cdot}7} = 19{\cdot}56 \text{ ft.}$$

The distances of the right-hand fixed points in the first and middle spans are, by symmetry, $b_1 = 19{\cdot}56$ ft. and $b_2 = 20{\cdot}06$ ft.

INFLUENCE LINES.—As a basis for computing the influence-line ordinates the bending-moment diagrams for unit concentrated load ($W = 1$) acting at each of seven equidistant positions in each span are plotted in *Fig.* 5. The closing lines of the diagrams have been determined by means of the values of $\frac{Y_L}{a}$ and $\frac{Y_R}{b}$ given in *Table* No. 1 in Part 1. Interpolation is unnecessary in this case; the coefficients for the value of n nearest to 0·11 (that is, $n = 0{\cdot}10$) in the part of the table for $2r = 2$ are used.

As an example of the calculation of one ordinate, assume the load acts at point (2) in the first of the three spans; from the table, $\frac{Y_L}{a} = -0{\cdot}3166$ and $\frac{Y_R}{b} = -0{\cdot}2408$, and, since a for this span is zero, Y_{L1} is zero (that is the fixed-point coincides with the support), and, with

$$b = 19{\cdot}56 \text{ ft.}, \quad Y_{R1} = -0{\cdot}2408 \times 19{\cdot}56 = -4{\cdot}71.$$

As a second example consider the load at point (9) in the middle span, which corresponds to point (2) in the table. Therefore

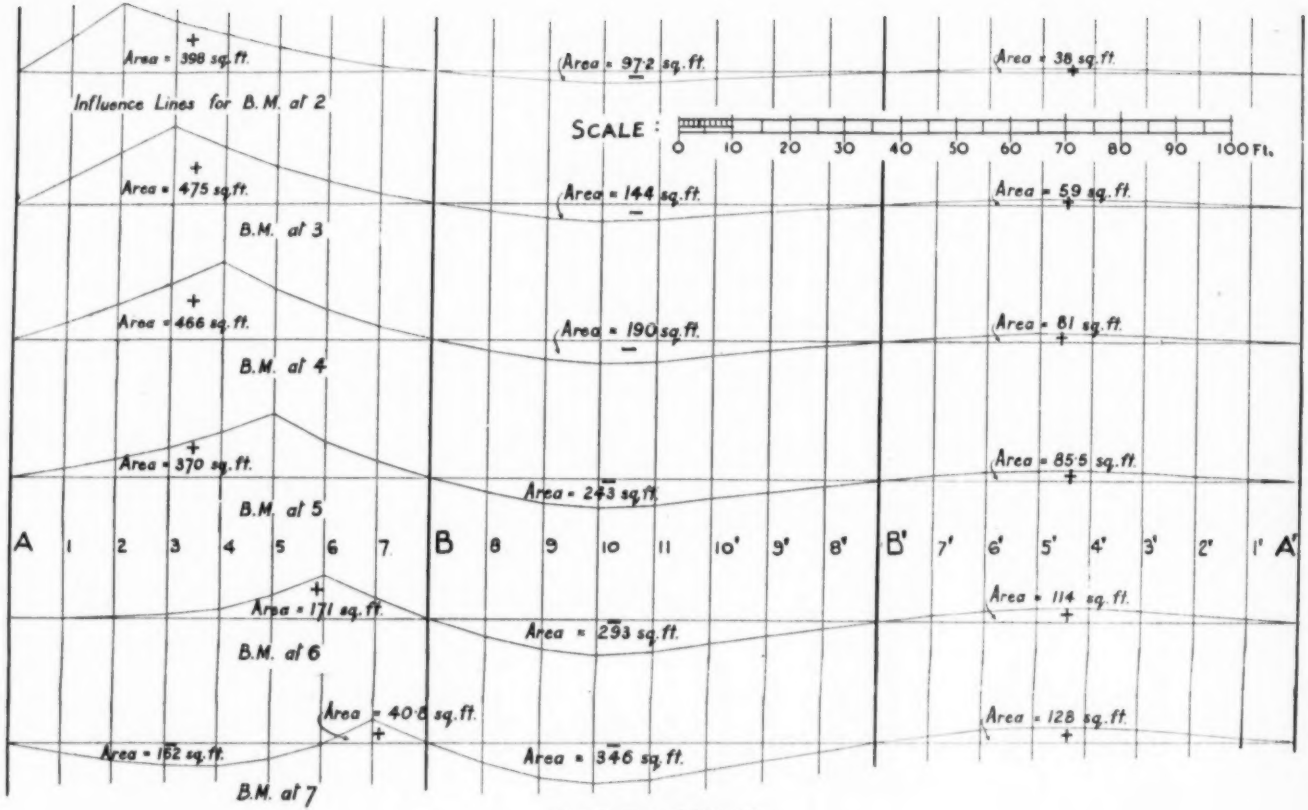
$Y_{L2} = -0{\cdot}3166 \times 20{\cdot}06 = -6{\cdot}35$, and $Y_{R2} = -0{\cdot}2408 \times 20{\cdot}06 = -4{\cdot}83$.

The influence lines for bending moments at points (1) to (11), and therefore at (1′) to (10′) because of symmetry, and influence lines for the shearing forces at the supports are given in *Figs.* 6 and 7, and are derived from the diagrams in *Fig.* 5 in a similar manner to those in the examples in the previous articles.

BENDING MOMENTS AND SHEARING FORCES.—The live load on the bridge is assumed to be a distributed load of $6 \times 220 = 1320$ lb. per linear foot, and a concentrated load of $2700 \times 6 = 16{,}200$ lb. acting in the most unfavourable position. The dead load is the weight of the slab, the wearing surface, and the rib of the girder, and is assumed to be 1470 lb. per linear foot.

The bending moments due to the dead load are determined directly, without the aid of the influence lines. The maximum free bending moments are:

In the first span: $\frac{1}{8} \times 1470 \times 76^2 = 1{,}061{,}300$ ft.-lb.
In the middle span: $\frac{1}{8} \times 1470 \times 81^2 = 1{,}205{,}500$,,


Influence Lines for B.M. at 2
Area = 398 sq.ft.
Area = 97·2 sq.ft.
Area = 38 sq.ft.
SCALE :
0 10 20 30 40 50 60 70 80 90 100 Ft.
Area = 475 sq.ft.
Area = 144 sq.ft.
Area = 59 sq.ft.
B.M. at 3
Area = 466 sq.ft.
Area = 190 sq.ft.
Area = 81 sq.ft.
B.M. at 4
Area = 370 sq.ft.
Area = 85·5 sq.ft.
Area = 243 sq.ft.
B.M. at 5
A 1 2 3 4 5 6 7 B 8 9 10 11 10' 9' 8' B' 7' 6' 5' 4' 3' 2' 1' A'
Area = 171 sq.ft.
Area = 114 sq.ft.
Area = 293 sq.ft.
B.M. at 6
Area = 40·6 sq.ft.
Area = 128 sq.ft.
Area = 152 sq.ft.
Area = 346 sq.ft.
B.M. at 7

[Upper Part of Fig. 6.]

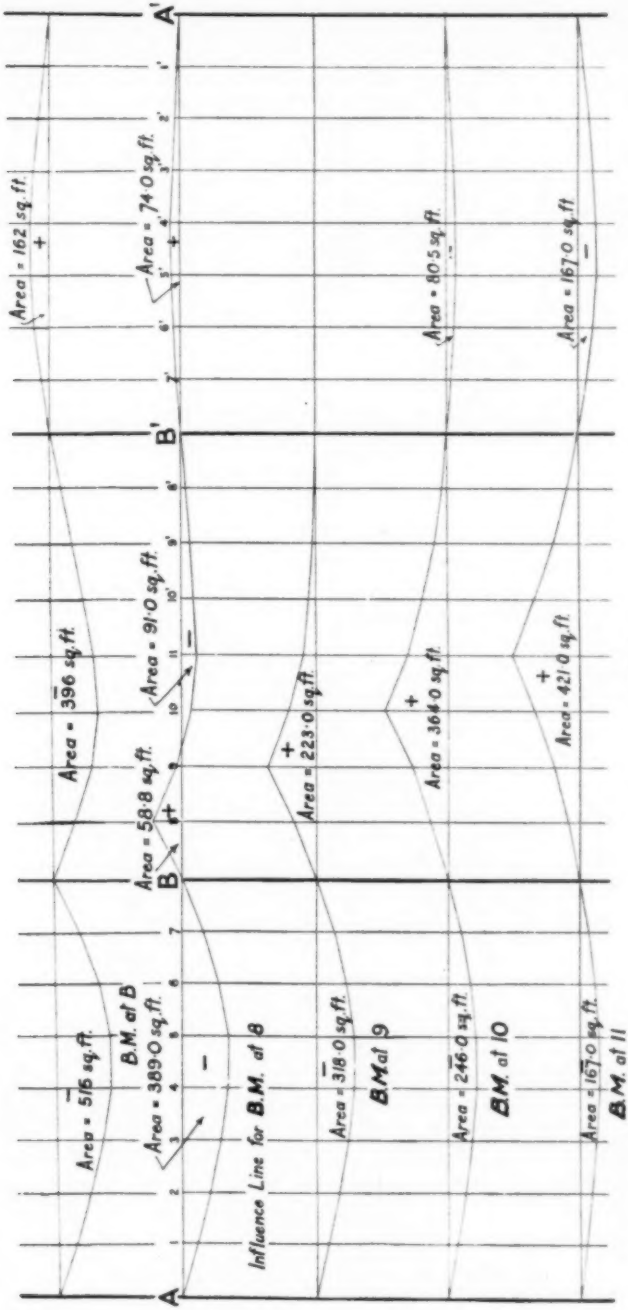

A
A'
B
B'
Area = 162 sq. ft.
Area = 74·0 sq. ft.
Area = 396 sq. ft.
Area = 91·0 sq. ft.
Area = 58·8 sq. ft.
Area = 516 sq. ft.
B.M. at 8
Area = 389·0 sq. ft.
Influence Line for B.M. at 8
Area = 223·0 sq. ft.
Area = 318·0 sq. ft.
B.M. at 9
Area = 364·0 sq. ft.
Area = 805 sq. ft.
Area = 246·0 sq. ft.
B.M. at 10
Area = 421·0 sq. ft.
Area = 167·0 sq. ft.
Area = 167·0 sq. ft.
B.M. at 11

Fig. 6.

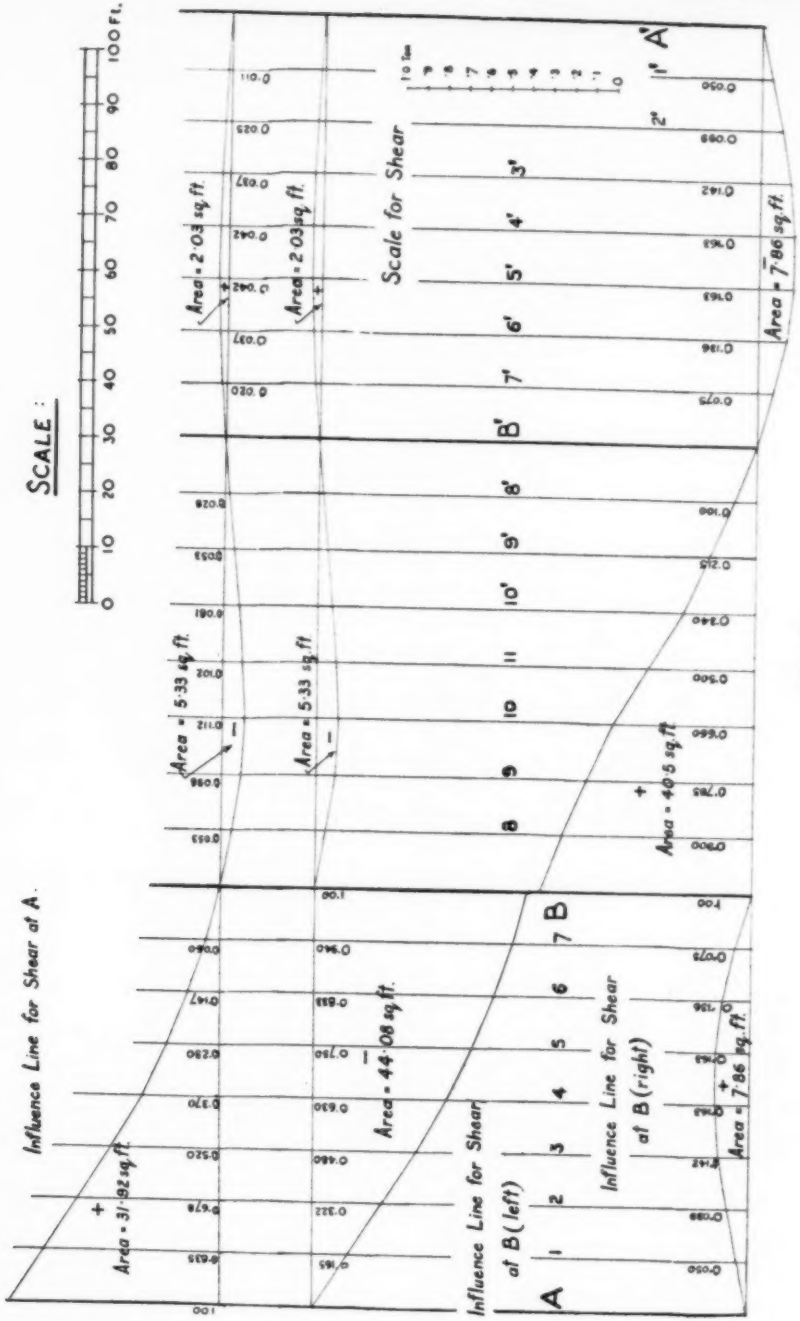

SCALE :
0 10 20 30 40 50 60 70 80 90 100 Ft.
Scale for Shear
10 Tons
Influence Line for Shear at A.
Area = 31·82 sq. ft.
Area = 2·03 sq. ft.
Area = 2·03 sq. ft.
Area = 5·33 sq. ft.
Area = 5·33 sq. ft.
Area = 44·08 sq. ft.
Influence Line for Shear at B (left)
Influence Line for Shear at B (right)
Area = 40·5 sq. ft.
Area = 7·86 sq. ft.
Area = 7·86 sq. ft.
A
B
A'
B'
1
2
3
4
5
6
7
8
9
10
11
1'
2'
3'
4'
5'
6'
7'
8'
9'
10'

Fig. 7.

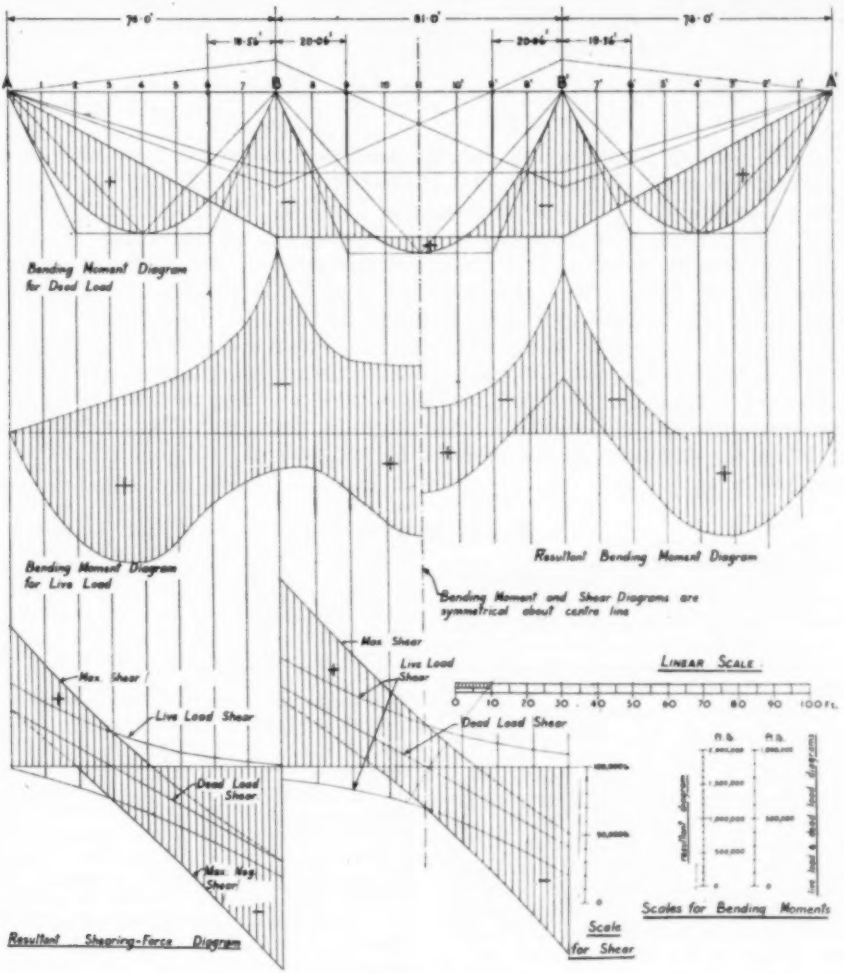

Bending Moment Diagram for Dead Load
Bending Moment Diagram for Live Load
Resultant Bending Moment Diagram
Bending Moment and Shear Diagrams are symmetrical about centre line
Max. Shear
Live Load Shear
Dead Load Shear
Max. Neg. Shear
Resultant Shearing-Force Diagram
Scale for Shear
LINEAR SCALE
Scales for Bending Moments

Fig. 8.

The parabolic bending moment diagrams are plotted in the upper part of *Fig.* 8 and, by the use of fixed points, the closing lines are determined by the method used in the example on page 83, March, 1948, and the resultant diagram shown shaded in *Fig.* 8 is obtained. The negative bending moment due to the dead load at support B is scaled from the diagram to be 1,070,000 ft.-lb. Therefore the shearing forces due to the dead load are:

At A. $\quad \frac{1}{2} \times 1470 \times 76 - \frac{1{,}070{,}000}{76} = 41{,}500$ lb.

To the left of B.

$$-\tfrac{1}{2} \times 1470 \times 76 - \frac{1{,}070{,}000}{76} = -69{,}700 \text{ lb.}$$

To the right of B.

$$\tfrac{1}{2} \times 1470 \times 81 = 59{,}500 \text{ lb.}$$

The bending moments and shearing forces due to the live load have been determined from the influence lines, and are plotted on the left-hand side of the lower part of *Fig.* 8. These diagrams are combined with those for the dead load, and the resultant envelopes of the diagrams for maximum positive and negative bending moments and shearing forces are obtained as shown in the lower part of *Fig.* 8. It is seen from the resultant bending-moment diagram that the greatest bending moment on the first and third spans is considerably greater than the greatest on the middle span. Where possible the middle span would be much longer than the end spans, which would be decreased correspondingly, and so the greatest positive bending moments would be more nearly equal; but in this example the length of each of the three spans, which form only a part of the whole structure, were determined by the most suitable positions for the foundations of the piers.

Book Reviews.

"Structural Theory and Design." By J. McHardy Young. Vol. 1. (London: Crosby Lockwood & Son, Ltd. 1950. 288 pp. Price 25s.)

IN this book, which is the first of two volumes dealing with structural design in steel, reinforced concrete, timber, and other materials, elementary theory is considered up to continuous beams and simple frames. The design of each structural member in different materials is considered collectively. In this way the student is taught to think of the member as the primary element, the principles of which are the same for all materials, while the differences in detail between members of different materials are emphasized. The author gives several methods of solving a problem and points out which is the most suitable for a given case. There are several numerical examples of a practical nature. Generally the descriptions are concise, but in some cases, such as the consideration of the resistance of reinforced concrete beams to shearing force, it is doubtful if the matter would be clear to a reader having no previous knowledge of the subject.

"'Hütte' Des Ingenieurs Taschenbuch." Vol. III, Part 2. Bauingenieurwesen. (Berlin: Wilhelm Ernst & Sohn. 1951. Price 14 D.M.)

THIS part of the 27th edition of this German standard reference book published on behalf of the academic institution "Hütte" has been completely revised. There are chapters on building construction, factory planning, heating and ventilation, road construction, town planning and plant installation. The chapter on building construction includes a summary of reinforced concrete practice, with many illustrations, and a survey of concrete work applied to buildings. In the section on road construction many useful details are given on problems which are apt to perplex the highway engineer, such as the adjustment of levels at a junction where one road is falling, or the arrangement of channel gradients and spacing of gullies on a flat road. The section on town planning gives examples of multiple-story garages with helical ramps. The chapter on plant installation includes diagrams of modern earth-moving machinery.

"Royal Festival Hall." (London: Max Parrish & Co., Ltd. 1951. 100 pp. Price 30s.)

AN official record of the Royal Festival Hall, London, published in association with the London County Council. It is illustrated by photographs, sketches, plans, and other drawings of the building. In addition to matters related to musical and other entertainment, there are descriptions in non-technical language of the foundations, construction, and acoustics.

The Design of Walls of Open Rectangular Tanks.

A PROBLEM in the design of open rectangular reinforced concrete tanks is the determination of the bending moments on a wall (*Fig.* 1) subjected to hydrostatic pressure and restrained by continuity at the bottom and on the two vertical edges, but unsupported along the top edge. A simple approximate solution, not well known in this country, is that given by J. F. Büchi in " Beton und Eisen " for April 20, 1938. Compared with a more exact analysis based on the theory of the bending of plates, the results are conservative but not wasteful. The following notes on Büchi's method are by Mr. S. Mayer.

The pressures resisted by the wall acting horizontally as a restrained beam and vertically as a cantilever are probably as shown in *Fig.* 2. For convenience of calculation the pressure diagrams are considered to be equivalent to a trapezium and a triangle as shown. If the intensity of horizontal pressure at the bottom of the wall is p_0 (*Fig.* 1), the maximum pressure resisted by horizontal beam action is αp_0 and by vertical cantilever action βp_0 (*Fig.* 2). The fact that the deflections of a horizontal and a vertical strip at their intersection due to the assumed pressures must be equal enables values of α and β to be derived, and the relations given by Büchi are

$$\alpha = \left(1 - \frac{m}{6 \cdot 2}\right)^4 \text{ and } \beta = \frac{m - 0 \cdot 3}{3}$$

in which m is the ratio $\frac{l}{h}$ of the length to the height of the wall (*Fig.* 1).

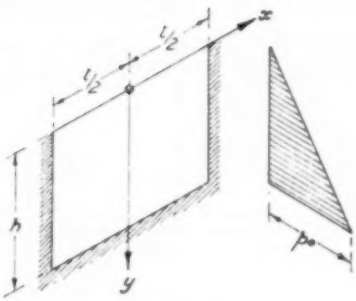


Fig. 1.

The maximum bending moment on a vertical strip of unit width is $\frac{\beta p_0 h^2}{6}$ or $\frac{\beta w h^3}{6}$ if w is the weight of unit volume of the liquid in the tank.

The maximum bending moment at the corners of a horizontal strip of unit width is $\frac{\alpha p_0 l^2}{12}$, that is $\frac{\alpha w m^2 h^3}{12}$, and occurs between $y = \frac{2}{3}h$ and $y = \frac{h}{4}$, depending on the value of m. At midspan the bending moment is about half this amount.

Fig. 3 shows typical bending-moment diagrams and indicates where the greatest bending moments occur. Complete fixity is assumed at the sides and bottom edges of the wall, a condition obtained in a tank square in plan and the base of which is not less thick than the wall. Modifica-

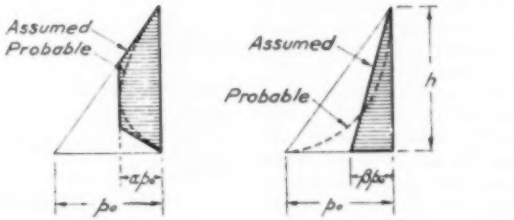


Fig. 2.

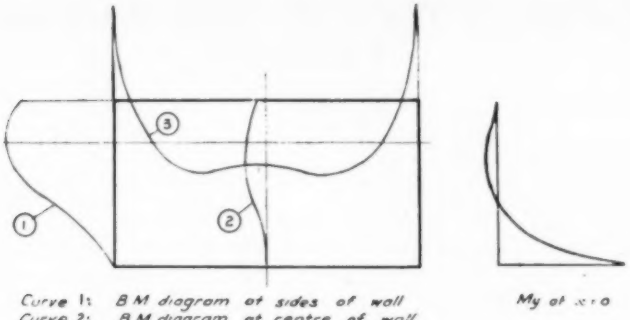

3
1
2
Curve 1: B.M. diagram at sides of wall
Curve 2: B.M. diagram at centre of wall
Curve 3: B.M. diagram at $y = \frac{h}{2}$.
My at x = 0
(a) Horizontal Bending Moments.
(b) Vertical Bending Moment.

Fig. 3.

tions can be made in other cases by considering that the effect of partial fixity at the sides is equivalent to increasing the value of m. It is suggested by Büchi that his method should be used only in cases where m is not less than 0·5 and does not exceed 2. Above and below these limits the wall should be designed to span respectively as a vertical cantilever ($\alpha = 0$, $\beta = 1$) and a slab spanning horizontally ($\alpha = 1$, $\beta = 0$), in both cases the full intensity of pressure being resisted.

For the condition of complete fixity, the various values of the maximum bending moment coefficients $\left(\frac{B.M.}{wh^3}\right)$ derived by Büchi's method and those derived by Professor Dana Young (Proceedings of the American Society of Civil Engineers, Nov., 1941), and Dr. Marcus ("Bauingenieur", 1936) are compared in *Table* I.

The horizontal tensile force in a wall of a tank, if the adjacent walls span partly horizontally and partly vertically, is less than if the adjacent walls span horizontally, because the reaction to the pressure resisted by vertical cantilever action is provided by the bottom of the tank. It is permissible to deduct from the horizontal force the amount resisted by vertical action as computed from the equivalent triangle in *Fig.* 2. Therefore the direct tensile force on unit width of the bottom of the tank is $\frac{\beta p_0 h}{2}$, or $\frac{\beta w h^2}{2}$. The tensile force to be combined with the maximum horizontal bending moment is $\frac{\alpha_1 p_0 l_1}{2}$ or $\frac{\alpha_1 w h l_1}{2}$, where α_1 and l_1 apply to the wall at right-angles to that being considered; for a square tank the horizontal tensile force is $\frac{\alpha w h l}{2}$ or $\frac{\alpha w m h^2}{2}$. This horizontal force should be considered to act over practically the whole height of the wall.

TABLE I

m	Bending moment	Coefficients $\left(= \frac{B.M.}{wh^3}\right)$		
		Büchi	Young	Marcus
0·5	$M_{vert.}$	0·0110	—	—
	$M_{horiz.}$	0·0148	—	—
1·0	$M_{vert.}$	0·0390	0·0353	—
	$M_{horiz.}$	0·0410	0·0288	—
2·0	$M_{vert.}$	0·0940	—	0·08236
	$M_{horiz.}$	0·0700	—	0·05017

Examples.

This method is applied in the following to the calculation of the maximum bending moments and forces on the walls of tanks containing water ($w = 62{\cdot}4$ lb. per cubic foot).

(*a*) A tank 10 ft. square and 5 ft. deep.

$$m = \frac{10}{5} = 2\,; \quad \alpha = \left(1 - \frac{2}{6{\cdot}2}\right)^4 = 0{\cdot}21\,;$$

$$\beta = \frac{2 - 0{\cdot}3}{3} = 0{\cdot}57.$$

Vertically (1-ft. strip).

The maximum bending moment is

$$\frac{0{\cdot}57 \times 62{\cdot}4 \times 5^3}{6} \triangleq 750 \text{ ft.-lb.}$$

The tensile force in the bottom is

$$\frac{0{\cdot}57 \times 62{\cdot}4 \times 5^2}{2} \triangleq 450 \text{ lb.}$$

Horizontally (1-ft. strip).

The maximum bending moment at the corners is

$$\frac{0{\cdot}21 \times 2^2 \times 62{\cdot}4 \times 5^3}{12} \triangleq 550 \text{ ft.-lb.}$$

The maximum bending moment at midspan is, say, 300 ft.-lb.

The tensile force to be combined with the bending moments is

$$\frac{0{\cdot}21 \times 62{\cdot}4 \times 2 \times 5^2}{2} \triangleq 330 \text{ lb.}$$

(*b*) A tank 9 ft. long, 6 ft. wide, and 5 ft. deep.

Longer walls. $m = \frac{l}{h} = \frac{9}{5} = 1{\cdot}8$; increase to, say, 2 to allow for partial restraint at the corners. Here, as in (*a*), $\alpha = 0{\cdot}21$, $\beta = 0{\cdot}57$, and the vertical bending moment and tensile force transversely in the bottom are also as in (*a*). The maximum horizontal bending moment at the corners (allowing for partial restraint only) is

$$\frac{0{\cdot}21 \times 2^2 \times 62{\cdot}4 \times 5^3}{\text{(say) } 15} \triangleq 430 \text{ ft.-lb.}$$

The maximum horizontal bending moment at midspan will be about the same.

Shorter Walls. $m = \frac{l}{h} = \frac{6}{5} = 1{\cdot}2.$

Reduce to, say, 1·0 to allow for restraint due to unequal length of adjacent walls.

$$\alpha = \left(1 - \frac{1}{6{\cdot}2}\right)^4 = 0{\cdot}49\,;$$

$$\beta = \frac{1 - 0{\cdot}3}{3} = 0{\cdot}23.$$

Vertically (1-ft. strip).

The maximum bending moment is

$$\frac{0{\cdot}23 \times 62{\cdot}4 \times 5^3}{6} \triangleq 300 \text{ ft.-lb.}$$

The tensile force longitudinally in the bottom is

$$\frac{0{\cdot}23 \times 62{\cdot}4 \times 5^2}{2} \triangleq 180 \text{ lb.}$$

Horizontally (1 ft. strip).

The negative bending moment at the corners is the same as for the longer walls, or

$$\frac{0{\cdot}49 \times 1^2 \times 62{\cdot}4 \times 5^2}{\text{(say) } 9} \triangleq 430 \text{ ft.-lb.}$$

The positive bending moment at midspan is almost negligible, but nominal resistance would be provided.

The horizontal tensile forces to be combined with the horizontal bending moments are :

In the longer wall :

$$\frac{0{\cdot}49 \times 62{\cdot}4 \times 5 \times 6}{2} \triangleq 460 \text{ lb.}$$

In the shorter wall :

$$\frac{0{\cdot}21 \times 62{\cdot}4 \times 5 \times 9}{2} \triangleq 300 \text{ lb.}$$

Reinforced Concrete Chimney 611 ft. High.

A REINFORCED concrete chimney completed at El Paso last year for the American Smelting & Refining Co. is claimed to be the tallest in the world. It is 611 ft. high from the top of the foundation, and has an internal diameter of 14 ft. at the top. At 13 ft. below ground level there is hard gravel which tests indicated would carry 8500 lb. per square foot. The foundation is 8 ft. thick, of which the lower 2 ft. 6 in. is an octagonal slab 66 ft. 6 in. wide across the flats, and the remaining 5 ft. 6 in. slopes inward to an octagon measuring 46 ft. across the flats at the top.

The outside diameter of the chimney is 44 ft. 10 in. at the bottom ; the wall is 9 in. thick at the top and 25 in. thick at the bottom. The chimney was designed and built by the Custodis Company.

A New Concrete Laboratory

THE new laboratory (*Fig.* 1) at the research station of the Cement & Concrete Association at Wexham Springs, Bucks, comprises principally a hall 30 ft. wide, 100 ft. long, and 26 ft. high for structural research and a room 30 ft. square and 14 ft. 3 in. high for research on precast concrete.

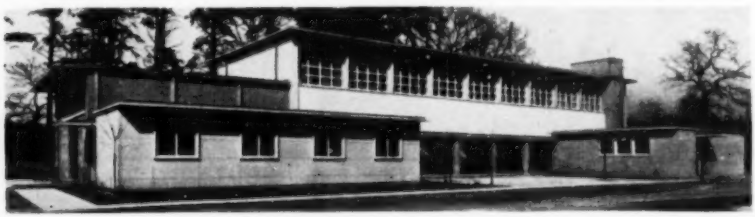


Fig. 1.—New Laboratories of the Cement and Concrete Association.

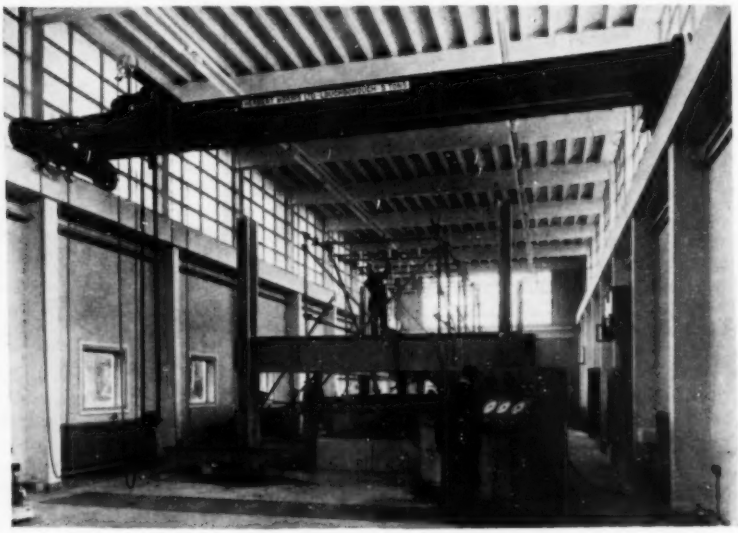


Fig. 2.—The Structures Laboratory.

The main building (*Fig.* 2) is a reinforced concrete frame structure, the tee-columns and roof main beams being at 10-ft. centres. The columns were cast in situ and support a 5-tons overhead hand-operated crane 15 ft. 5 in. above the floor. The main beams, which span 29 ft. 6 in., were precast and are prestressed with bonded wires. The ends of the beams and the heads of the columns are connected by ½-in. bars projecting from the beams and columns and embedded in a block of cast-in-situ concrete monolithic with the flat overhanging eaves. Precast prestressed concrete roof joists of tee-section span between the main beams, and the filling between the joists is of plain concrete blocks. The roofs of the precast concrete laboratory and offices are 4-in. cast-in-situ reinforced concrete slabs.

The floor of the precast concrete

laboratory and the test floor of the structures laboratory are isolated from the main structure by strips of felt impregnated with bitumen so that movement is unrestrained and the transference of vibration is prevented. The test floor (*Fig.* 3) is 40 ft. by 27 ft. by 2 ft. 6 in. thick and is reinforced with two layers of 1-in. bars at 6-in. centres in the bottom and at 21-in. centres in the top. Twenty-four ducts of 1½ in. diameter are provided at 12-in. centres in the slab. Generally the ducts are 20 in. below the top of the slab but rise in a catenary to within 6 in. of the top at the ends. Cables can be threaded through the ducts so that the floor can be prestressed if necessary to increase the resistance to sagging. Channels at each end provide space for the jacks. Threaded sockets, of which there are 263, arranged on an equilateral-triangular grid and connected to hook-bolts which interlock with the lower layer of reinforcement, are embedded in the floor and enable the testing frame or specimens to be fixed in any position. Each socket can resist a pull of 5 tons. The floor is waterproofed with bitumen 1¼ in. thick horizontally and ¾ in. thick vertically.

The walls of the main building are carried on ground beams spanning between the foundations of the columns. The north wall is of brick. The south wall (*Fig.* 1) is of 18-in. by 9-in. by 9-in. hollow concrete blocks. The window frames are of precast concrete. The surface finish of the walls demonstrates many of the colours and textures obtainable with cement rendering and different treatments of concrete, including coloured rendering, precast slabs of different textures, reconstructed stone, and concrete bricks. Corrugated asbestos-cement sheets are used on part of the south front.

The Testing Equipment.

Concrete specimens for test will be cast at one end of the structures laboratory and the instruments and apparatus are stored, and erected where possible, at the other end, where there is a 50-tons strength-testing machine. Main tests are carried out on the test floor and testing frame in the centre of the laboratory. The frame is arranged for use with hydraulic jacks. Holes in the steel frame enable the steelwork to be assembled to suit different specimens, and other members can be added. The frame can be erected to enclose an experimental structure so that concentrated or distributed loads can be applied directly. It can also be used for testing columns in direct compression, racking tests on frames, torsional tests, and tests on large beams. In *Fig.* 4 the frame is arranged to test interconnected prestressed beams. The six double-acting hydraulic jacks are fed by an axial plunger pump having a capacity of 938 cu. in. per minute at pressures up to 4000 lb. per square inch.

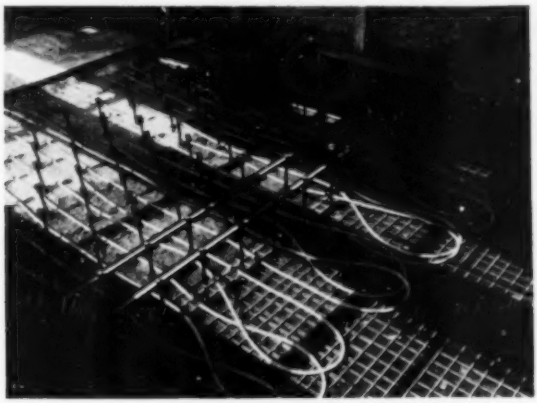


Fig. 3.—Test Floor during Construction.

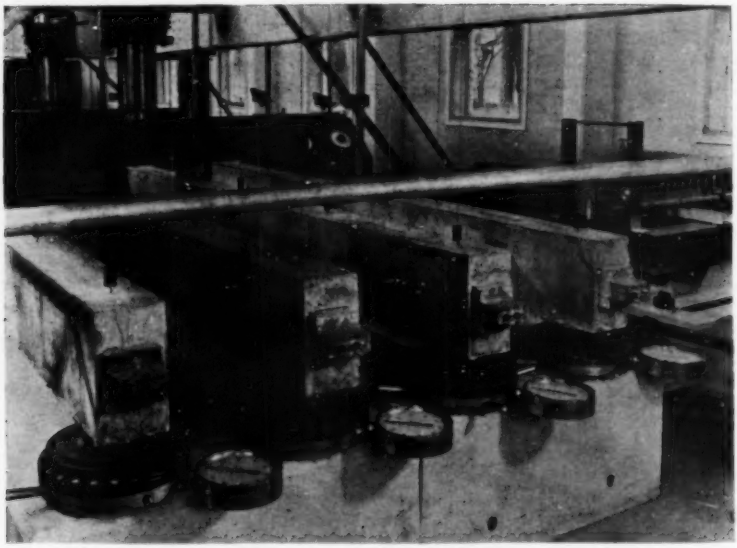


Fig. 4.—Prestressed Beams under Test, showing Pressure Capsules.

The pump is driven by a 10-h.p. motor. The jacks can exert a push of 50 tons or a pull of 21 tons. The forces and reactions are measured by 50-tons pressure capsules (*Fig.* 4), the moving and fixed parts of which are joined by bonded rubber. During a test on a capsule with the load on the jack acting at 5 deg., slight tilting only of the moving part was caused, and the capsule measured accurately the normal component of the load applied by the jack. All controls and gauges are in a mobile cabinet (*Fig.* 2). Loads can also be applied to a specimen by weights or by pneumatic bags placed between the specimen and the frame. Strain measurements will be made by electrical-resistance apparatus, acoustic gauges, and mechanical (dial) indicators.

The architect was the late Christopher Nicholson, M.A., the associate architects being Mr. Hugh Casson, M.A., and Mr. Neville Conder. The engineering consultant is Mr. W. A. Fairhurst, and the main contractors were Messrs. Holland & Hannen and Cubitts, Ltd.

A Patent Method of Consolidating Ground.

GROUND, masonry, and the like is consolidated, sealed, or filled by forcing into it one after the other two different salt solutions which, on encountering each other, form voluminous sticky and nearly water-insoluble compounds. The first solution is a mixture of waterglass with a solution of an alkali phosphate, preferably trisodium phosphate and/or alkali soap, and may also include concentrated alkali carbonate solution, for example sodium carbonate solution. The second solution may be of a soluble salt, preferably the chloride of calcium (in particular), magnesium, barium, beryllium, zinc, cadmium, aluminium, lead, or iron. The salt solutions employed may be dilute, saturated, or super-saturated, and the waterglass solution should have an $Na_2O : SiO_2$ ratio of at least 1 : 3·2 by molecular proportion. The first solution preferably contains not more than 0·6 part by volume of saturated alkali phosphate or alkali soap solution or solution mixture to 1 part by volume of concentrated waterglass solution. The soap solution used may be resin soap prepared by dissolving common resin in a dilute solution of caustic soda and decanting when clear.—Patent No. 605,154.—A. G. Guttmann. December 17, 1945.

Engineering Conference in London.

THE following are abstracts of some of the papers read at the Joint Engineering Conference held in London last month by the Institutions of Civil, Mechanical and Electrical Engineers.

Power Stations.

"Power Station Design and Construction", by V. F. Bartlett.—In the construction of modern electricity generating stations the cost of the civil engineering and building works is about 25 per cent. of the total cost. Economy of structural design is therefore important, as is also speed of construction since the capital charges during the construction period are great. An economical form of construction for the walls and roof of a power station is a series of reinforced concrete two-pin arches over the turbines and boilers. The arch ribs would require to span 390 ft. and have a rise of 188 ft. above the floor. The cross-section would be 7 ft. by 2 ft. and the ribs spaced at 16 ft. centres. The roof covering could be precast reinforced concrete slabs. The estimated cost compares favourably with ordinary structural steel and brick construction, but the main advantage would be the speed of erection and the lack of restrictions in the arrangement of the generating plant.

Decks of Railway Bridges.

"Railway Bridges", by J. S. Campbell.—A new type of reinforced concrete deck (*Fig.* 1) supported on welded steel girders, as used for the reconstruction of bridges destroyed by floods in Scotland in 1948, is described. The "well" deck forms a container for the ballast and acts as a guard-rail. The deck is in two parts, each of which carries one track, thus enabling one track to be put into use while the construction of the other part is in progress. A standard reinforced concrete slab and beam construction has been developed by British Railways for spans from 10 ft. to 50 ft.; it is economical and easy to erect without long interruption of traffic, and is economical to maintain.

Several precast prestressed concrete

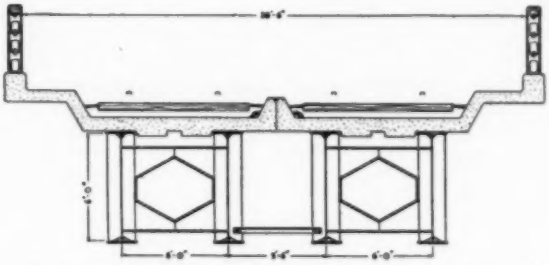


Fig. 1.—"Well" Deck for Railway Bridge.

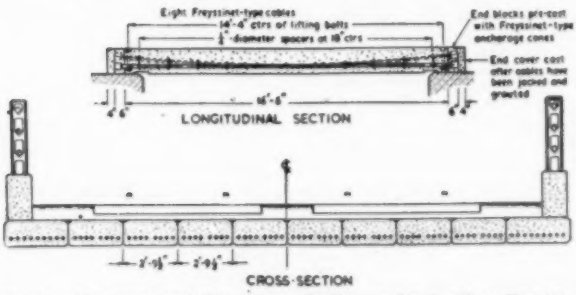

Eight Freyssinet-type cables
16'-4" ctrs of lifting bolts
3" diameter spacers at 18" ctrs
End blocks pre-cast with Freyssinet-type anchorage cones
End cover cast after cables have been jacked and grouted
16'-6"
LONGITUDINAL SECTION
CROSS-SECTION

Fig. 2.—Prestressed Concrete Railway Underline Bridges.

slab bridges (*Fig.* 2) have been built within the past 18 months to carry main lines. The spans are from 10 ft. to 20 ft. and the slabs are either 5 ft. 7 in. or 2 ft. 9½ in. wide. The Freyssinet method of stretching the cables after the concrete has hardened is used generally for the reconstruction of bridges because the design allows for variations of dimensions and conditions which cannot be avoided. When it is possible to use a large number of identical slabs, the method in which the wires are stretched before the concrete is placed around them is considered. The use of prestressed concrete by British Railways is not the result of the fact that bridge engineering may be passing through a prestressed-concrete phase, but because, among other factors, in first cost and maintenance costs prestressed concrete construction is cheaper than steel construction and the saving in steel is 86 per cent. compared with joist-in-concrete and 75 per cent. compared with reinforced concrete.

Dry Docks.

" Dry Docks ", by J. Guthrie Brown.—The author considers the design of thirty-two large dry docks. The main development is the gradual reduction in the number of altars. In docks used by naval vessels, such as the Captain Cook dock at Sydney and the Sturrock dock at Capetown, which were described in this journal for February and November, 1946, the walls are vertical and have two steps in their height with two additional altars formed by shelves cantilevered from the vertical faces. In docks used solely by merchant ships, altars are omitted as in the King George V Dock at Southampton, the faces of the walls of which are battered slightly to permit access of light, and massive vertical buttresses project at intervals from the walls. A vertical wall results in a reduction of material and excavation as well as a considerable saving in pumping. Further saving in the quantity of material is obtained by forming subways in the walls for services and culverts. Costs are reduced also by substituting high-grade concrete for masonry.

Runways.

" Planning and Construction of Airfields ", by J. A. Dawson.—The author stresses the need for reliable knowledge of the ground on which a runway is to be laid. Different types of pavement are discussed, especially the rigid concrete slab which is principally used in Great Britain, although this type is not necessarily the most suitable for heavy aircraft. There is no advantage in using rigid construction on a subgrade the bearing capacity of which is equal to or exceeds the intensity of pressure of the wheel. With ground of low bearing capacity it is likely that the thickness of a flexible base required to spread the load is so great that it is more economical to use a rigid slab which can distribute the load without requiring a deep sub-base on the ground. The strength of the ground is not capable of exact evaluation and, although the formulæ of Professor Westergaard for the design of rigid slabs have been of value, it is possible that the tests being made by the Air Ministry may prove to be a more reliable basis of economical design.

It is the practice in Great Britain to provide expansion joints at intervals of not more than 120 ft., but the distance between such joints in America is sometimes at least five times as much. The correct spacing of contraction joints is essential to prevent uncontrolled cracking and is dependent on the thickness of the slab; a spacing of 20 ft. is suitable for 12-in. slabs, and less for thinner slabs. In a test at London Airport, 8-in. and 10-in. concrete slabs laid on a concrete base, with construction and contraction joints 20 ft. apart, cracked spontaneously into four parts without being subjected to loads. In Great Britain, with few exceptions, steel dowels to transfer load across a joint are not used, the reasons being scarcity of steel, some lack of confidence in the method, and the possible influence of the steel on the efficiency of electrically-operated ground control of aircraft. In consequence, double-slab construction has been developed. An upper slab of high-grade concrete is laid on a concrete base of low-strength concrete, the two slabs being separated by a bituminous layer. The joints in the upper slab do not coincide with the joints in the lower slab. A corner of the upper slab, in this construction, is several times stronger than a corner of the same thickness resting on the ground. In British practice concrete runways are not generally reinforced, but concrete of very high quality is used.

Engineering Education.

THE following paragraphs are abstracts from a paper entitled " The Development and Trend of University Education in Engineering," presented by Professor J. F. Baker, O.B.E., M.A., Sc.D., at the Joint Engineering Conference held in London last month.

A report in 1832 of the British Association on the state of knowledge of hydraulics, says " It only remains for us to notice the scanty contributions of our own countrymen. While France and Germany were rapidly advancing on the traces of Italy, England remained an inactive spectator of their progress." In fact, at this time, much of the engineering theory now taught as a matter of course was unknown or barely understood, especially in Britain.

A successful practising industry existed long before university education in engineering, and as late as 1870 a report on the Education of Civil Engineers, produced by the Institution of Civil Engineers, could say " It is not the custom in England to consider theoretical knowledge as absolutely essential." In 1891 the calendar of Mason's College advertised " The whole of the teaching in the classes is practical and technical in its aim, no theory being introduced except such as has direct bearing on the problems of professional practice, and no theories being taught until the facts upon which they are based have been fully explained." When Birmingham University was founded in 1900 it could be said that " The over-riding need was considered to be the preparation of men capable of filling positions in industry with a minimum of further training, and therefore bias was given towards the more practical side of engineering." At that date, with few exceptions, all universities had workshop courses, with methodical instruction in the use of hand and machine tools.

In 1921 two universities—Liverpool and Sheffield—increased their courses to four years post-matriculation or three years post-intermediate, and delayed specialization till the final year, so that a broader scientific background was received by their undergraduates. Between the two wars some universities reorganized their courses to conform, with minor variations, to this pattern, but a substantial proportion delayed the change until after the second world war. On the other hand, Edinburgh, which until then had only one course in engineering, permitted, in 1924, specialization in the final year. Only Oxford and Cambridge persisted in giving an all-embracing course to a high level, in the firm belief, as Professor Inglis said in 1931, that such graduates " possess a versatility and balance of mind which enable them to strike out with equal facility along any line they may subsequently be called upon to pursue. Premature specialization cramps the imagination and is destructive of the length and breadth of mental vision." One is forced to the conclusion, however, that as the syllabus grows only the ablest men can carry a large number of subjects to a high level simultaneously, and the alternatives are a lowering of standards or specialization.

Some universities are questioning the value of formal experiments in the laboratory and are introducing small investigations and projects in the final year as a means of encouraging initiative and a methodical approach to new problems. Similarly, alarm at the inability of students to think and write logically has led, in one case, to formal instruction in the presentation of technical information and in others to the use of discussion groups or to practice in the writing of essays and long reports. Efforts are also being made to encourage undergraduates to achieve a more general education, not by the pathetic step of adding a smattering of classical culture which should have been acquired at school, but by showing the relationship of engineering to other sciences and arts. This development may represent the beginnings of a new attempt to strike a balance between belles-lettres, mathematics and experimental philosophy, which was attempted so long ago as 1807 by Belfast Academy and which was referred to by the Principal of Owen's College in 1873 when he said of university education " The subject matter of the studies selected is, in fact, of less importance than the discipline imparted. But a selection must be made which will draw out and strengthen the powers of the mind and afford a broad basis on which to build a subsequent professional career. The common groundwork must therefore include (*a*) Letters—

to cultivate taste and judgement and to give style in speech and writing; (*b*) Mathematics—to discipline the reasoning faculty and to give the habit of concentrated thought; (*c*) Some branch of physical study to develop the powers of observation and inductive reasoning, and to impart the method of this study, thus enabling a student to provide himself with pleasure and facility, a knowledge proper to his calling."

The following points were made by Mr. H. J. B. Harding, B.Sc., in a paper on "The Practical Training of Civil Engineers" presented at the same conference.

The handing down of experience appears to follow a recurring pattern. The first master passes on his experience to the second generation who has partly shared it with him; it is then passed to the third, who knew not the first, with all the sanctity of the original truth; he passes it to the fourth as a rigid code, without much appreciation of the reasons behind it, but he himself is protected from seeing the effects of disobedience; the fifth, however, may come to questions and deride the old-fashioned concept and ignore the teachings; he then re-learns by bitter experience the original truths discovered by number one.

It should be remembered, when reading of the Tennessee Valley schemes, that the British built equally large and important works in North-West India, which are now forgotten or unknown to the present generation. Just before the second world war, the tractor and heavy excavation plant were introduced from America, in time to participate in the enormous airfield construction programme. These airfields provided the greatest outburst of one type of work since the railway mania. The tractor and scraper became a fetish from which recovery is but slow, and this type of work got rather out of proportion in its relation to training and practice.

Bazalgette's Victoria and Chelsea embankments, which, starting 100 years ago, revolutionized the appearance of London, included part of his magnificent main drainage scheme, which was—and still is—one of the unseen wonders of the world. This work, opposite the Festival site, is taken entirely for granted by the Londoner, and appears to be quite ageless and undated.

The function of a university is to provide facilities for research by the teaching staff. In civil engineering there is room for more full-scale research on the construction site and less in the laboratory, and members of university staffs should have opportunities to carry out such research. They could be attached to works as were Royal Engineer officers and, while still wholly or partially paid by their colleges, they could do useful work on sites where the normal staff were not free to give their minds and time to research, and would return to their studies with minds refreshed by contact with reality. This could apply to engineers in research organizations.

The Reinforced Concrete Association.

THE annual meeting and lunch of the Reinforced Concrete Association was held in London last month. Mr. J. Cuerel was elected president for the ensuing year. The retiring president (Mr. A. R. Neelands) stated that there were now 106 Members and 301 Associates of the Association. Referring to the difficulty of obtaining steel reinforcement, Mr. Neeland said that the production of mild steel reinforcement bars and wire had increased this year by 500 tons a week, but this was not enough to meet the demand. He estimated the present shortage of steel reinforcement as one-third of the total requirements, and considered that the deficiency would become worse unless production was further increased.

Reinforcement in Slag Concrete.

A FIVE-SPAN open-spandrel concrete arch bridge at Akron, Ohio, was recently demolished. It was built in 1913, and was one of the first bridges in the United States to be built of concrete with blast-furnace-slag aggregate. Examination of the reinforcement (twisted square bars) after the demolition showed the bars to be in good condition after being buried in slag concrete for more than 35 years.

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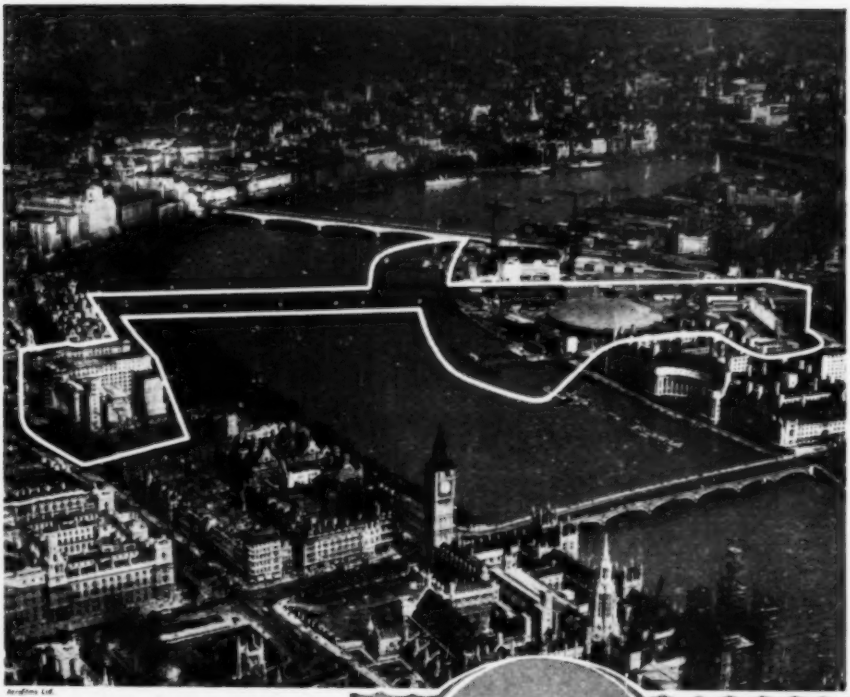

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New Government Offices at Whitehall Gardens, Festival entrances and foundations for Bailey Bridge, reconstruction of the South Bank River Wall and all the Festival Buildings in the upstream area, are a group of important contracts being completed in London.

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Shuttering for Aerated-Concrete Walls.

THE aluminium-alloy shuttering used by Messrs. George Wimpey & Co., Ltd., for the erection of walls of aerated (or foamed) concrete is shown in *Figs.* 1 and 2. This type of construction is called "Situfoam," for which almost any type of sand from coarse desert sand to wind-blown dune sand may be used. Several hundreds of dwellings have been constructed by this method in Kuwait (Persian Gulf), Borneo, and Sumatra. The foamed concrete has a weight of about 90 lb. per cubic foot and a crushing strength sufficient for dwellings. The thermal insulating value of a 9-in. wall is slightly better than that of an 11-in. hollow brick wall and about equal to that of an 18-in. solid brick wall. The concrete is of liquid consistency and is pumped through pipes and flexible hoses of 3-in. and 4-in. diameter. Generally the coarser the sand the easier the pumping. The mixer, aerator, and pump are mounted on a platform fitted with wheels or skids to facilitate movement about the site.

The shuttering comprises planks and walings of extruded aluminium alloy, which are connected by light alloy clamps (*Fig.* 2) and greased tie-bolts which pass through the wall. The widths of the planks are such that a variation of 3 in. in any direction on plan is possible. The height can be varied to suit requirements. An accuracy of $\frac{1}{8}$ in. is obtained on lengths of shuttering up to 40 ft. One man can carry with ease a piece of about 20 sq. ft. The shutter illustrated in *Fig.* 1 is 38 ft. long and 10 ft. high and is in one piece. The largest shutter used so far in one piece is 54 ft. long and 11 ft. high and weighs about 22 cwt.

As the concrete is placed to the full depth of the shutter in one operation, the pressure of the concrete is considerable. In designing the shuttering, the pressure is calculated by the Rankine formula assuming an angle of repose of 5 deg.

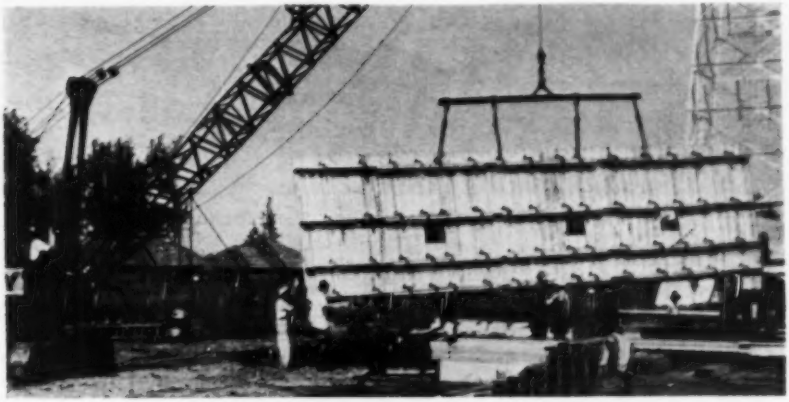

Fig. 1.—First Stage of Erection of Wall Shuttering.

The tensile force on each of the lower tie-bolts, which are usually at about 3-ft. centres for a wall 10 ft. high, exceeds 2 tons. Walls up to 12 ft. 6 in. high have been cast without visible deflection of the shuttering. Some shutters have been in constant use for more than two years, and few replacements have been necessary. The alloy has a high resistance to corrosion. A hard impenetrable face of oxidised material forms on exposed surfaces and produces an excellent finish to the concrete. The face is preserved by prohibiting the use of metal cleaning tools.

The shutters are generally lifted from one dwelling to the next by a mobile crane (*Fig.* 1), but movement and erection by hand are also practicable. When the inner shutters for the entire structure,

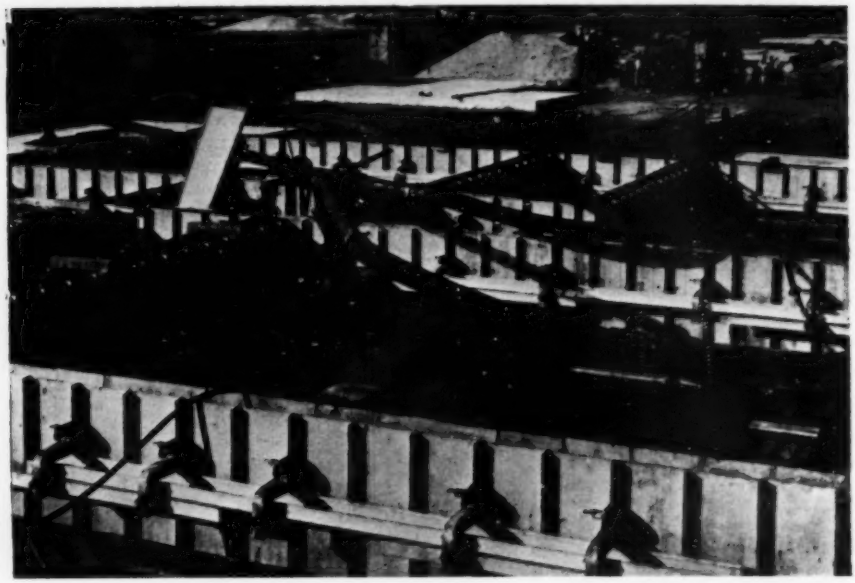

Fig. 2.—Shuttering and Concrete Pipeline.

including the partitions, are erected, door and window frames, electric conduits, cores for small openings, etc., are fixed to the shuttering. The outer shutters are then placed in position and secured to the inner shutters by the tie-bolts and clamps. A working platform (*Fig.* 2) is erected near the top of the shuttering and may be supported on the walings or directly from the floor. The whole of the interior and exterior walls are cast in one day. Due to the fluidity of the contrete no tamping is required and no air-pockets are formed under window-sills. The shutters are removed the following day and re-erected for the next block of dwellings. At a site in Sumatra, three sets of shutters were used and nine pairs of semi-detached dwellings were constructed weekly. The surface of walls as left by the shuttering is sufficiently good to be distempered without plastering. The outside faces of walls are usually given two coats of white cement paint.

Professional Announcement.

Mr. G. P. Manning, M.Eng., M.Inst.C.E., of 5 Gunnersbury Gardens, London, W.3, is now in practice as a consulting engineer in association with Mr. R. A. B. Smith, M.C., A.M.Inst.C.E.

Change of Address.

THE head office of Blaw Knox, Ltd., is now at 90–94 Brompton Road, London, S.W.3. Telephone: Kensington 5151; telegraphic address: Blawnox Southkens London.

Competition.

THE sending-in date for the competition for ideas for improving efficiency in the building trades, organised by Acrow (Engineers), Ltd., has been extended from July 2 to July 31.

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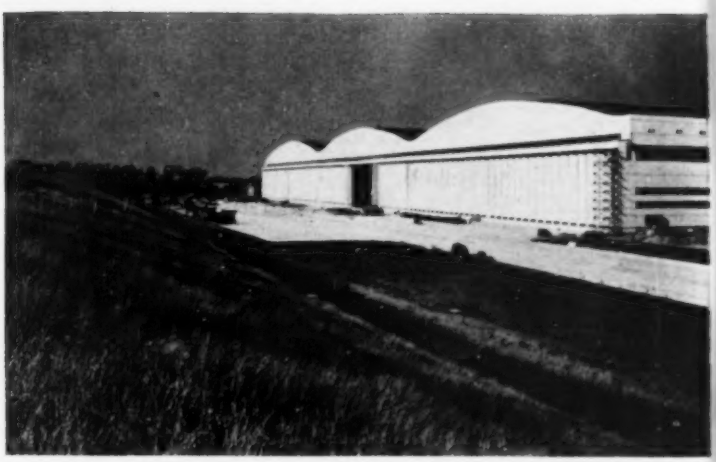

" Ritecure " membrane curing material forms a cellophane-like skin on the surface of the concrete. Since it is applied before the setting of the cement has taken place, it ensures retention of the maximum amount of moisture in the concrete, eliminating hair and wind cracking caused by too rapid drying out of the cement in the face of the concrete immediately after laying. It is not affected by the heat of the sun and, while it is not sold as an anti-frost mixture, since " Ritecure " itself will not freeze, it gives protection against 5–10° of frost.

This one-man operation of curing concrete—horizontal or vertical surfaces—has brought concrete curing to a simple, speedy and economical process, whereby the highest degree of curing efficiency is obtained without the use of costly covering materials and their heavy handling charges. It is easily applied through a spray, and various sizes of equipment are available for small or large areas. " Ritecure " is in use in temperatures of extreme degrees all over the world, and is being extensively used by leading Engineers and Contractors in this country for concrete runways, roads, and other forms of concrete work.

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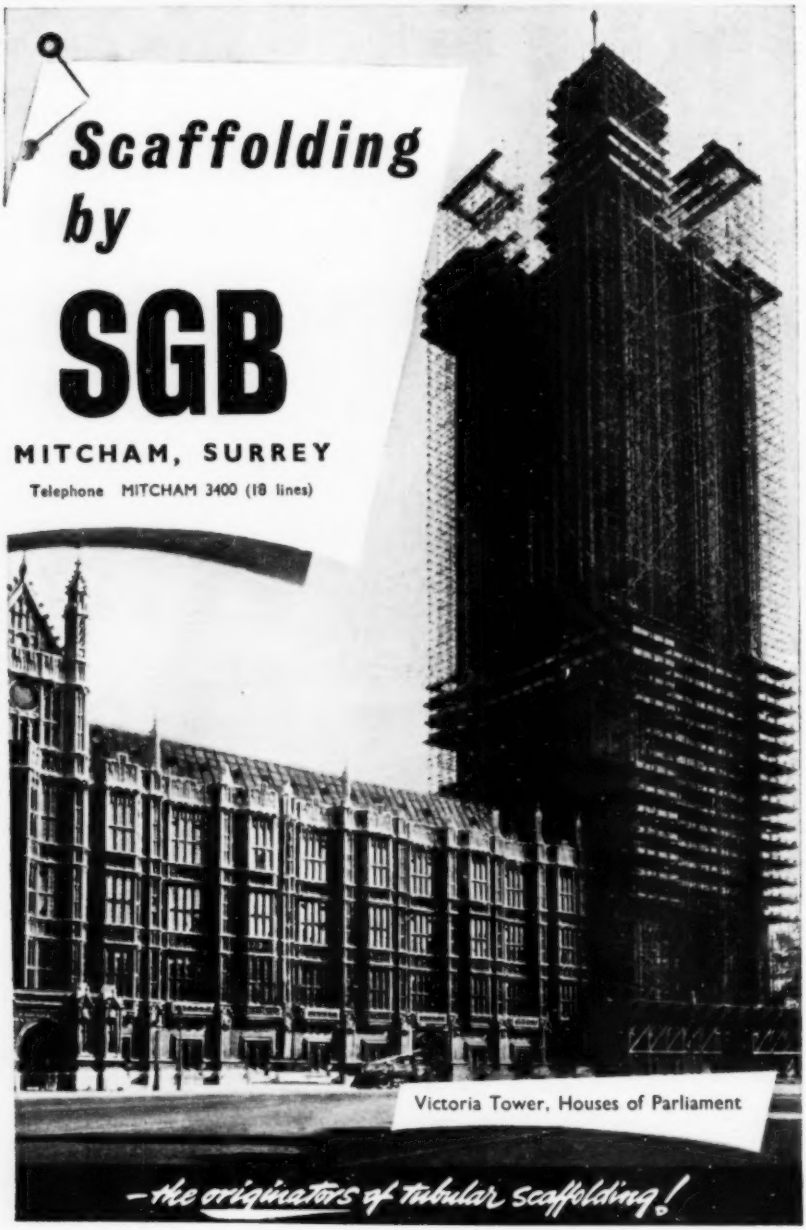

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Machine for Roughening Concrete Surfaces.

THE machine shown in (*Fig.* 1) is designed for roughening horizontal concrete surfaces by removing, in the form of dust and fine chippings, the surface for a depth of about $\frac{1}{16}$ in. to $\frac{1}{8}$ in. The machine is mounted on a two-wheeled hand trolley, and comprises four drills having tungsten-carbide steel bits (*Fig.* 2) operated pneumatically by independent cylinders and caused to rotate and at the same time to have a percussive action. The cross-bar to which the drills are attached can be moved backwards or forwards along the frame carrying the road wheels so that the weight bearing on the concrete surface can be increased or decreased. The operator pushes the machine slowly forward with a weaving motion so that the drills traverse the whole of the area under the machine and roughen a strip about 2 ft. wide. The roughness of the surface and the depth of the cut can be varied by using different bits. The average area that can be roughened in an hour is 10 to 12 sq. yd. For narrow strips a smaller machine with two bits is used. *Fig.* 3 shows a concrete slab which has been roughened on one side only of a joint, the edges of which are not damaged.

Fig. 1.

Fig. 2.

Two men and an air-compressor are required to operate the machine. A

machine with four bits may use about 180 cu. ft. of air per minute and one with two bits about 90 cu. ft. The bits require to be sharpened when about 200 sq. yd. of gravel concrete have been roughened, and it is necessary to replace a bit after it has been sharpened five or six times.

The machine may be used to roughen the surface of concrete to form a key for granolithic or other monolithic finishes. To demonstrate the efficiency of the key, specimen ½-in. toppings of granolithic were laid by Metropolitan Construction Co., Ltd., who developed the machine. Some toppings were laid on an old concrete surface which had been brushed and washed with detergents, and others on the same concrete after the surface had been roughened by the machine. The latter toppings were subjected to repeated blows by a sledge-hammer without any signs of disintegration of the topping or severing of the key. Similar treatment applied to the granolithic laid on the brushed surface quickly caused the topping to crack and break off in large pieces. The machine may also be used to form a key when a granolithic finish is to be laid on a new concrete floor some time after the floor is laid. The machine has also been used for roughening concrete roads and runways that have been worn smooth by traffic. The coefficient of friction of the roughened surface may be up to 0·7.

Fig. 3.

Depositing Concrete in Road Construction.

FULL-SCALE tests made to determine the effect of different methods of depositing concrete when constructing roads are described in " Roads and Road Construction " for January 1951, by Mr. R. H. H. Kirkman of the Road Research Laboratory. The least irregularity of the finished surface was obtained when small heaps of about 1 cu. yd. of concrete were dumped from both sides of the slab at regular intervals. When concrete was dumped from one side only the irregularity on this side of the slab was considerably greater than on the other, and it was necessary to place heaps fairly close together. Honeycombing at the bottom of the slab was detected when heaps slightly overlapped.

When the spreading was poor, the riding quality of a level road was improved by substituting concrete of low workability for the usual concrete of very low workability. When the superelevation was great the flow of the wetter concrete considerably increased the irregularity. The wetter concrete was also difficult to handle and appeared to lead to honeycombing at the bottom of the slab. The removal of irregularities became increasingly difficult with each pass of the finishing machine. The best riding quality was achieved when irregularities were detected after the first pass so that an attempt could be made to remove them during the second pass.

Even with high-quality workmanship and good supervision, uneven setting and settlement of the side forms was responsible for much irregularity.

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SITUATIONS VACANT.

SITUATIONS VACANT. The Liversedge Reinforced Concrete Engineering Co., Ltd., Liversedge House, John Adam Street, London, W.C.2, invite written applications for the following positions in their London and Brighton offices: (i) Designer and designer-detailers with particular experience in arch shell roof design and detailing. (ii) Detailers with design knowledge, and draughtsmen for work on all types of reinforced concrete structures. (iii) Keen juniors with technical ability for training. State full details of qualifications, age, and summary of experience.

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SITUATION VACANT. A well-known firm of heavy industrial plant engineers, situated near Manchester, requires a reinforced concrete designing engineer with experience in all types of industrial structures. The position is a permanent one and progressive. A pension fund is in operation. Five-day week. The commencing salary will be between £800 and £1,000 a year according to ability. Apply briefly stating previous employers, with dates and positions held, to Box 2476, CONCRETE AND CONSTRUCTIONAL ENGINEERING, 14 Dartmouth Street, London, S.W.1, and quoting reference PE/707.

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(Continued on facing page.)

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(*Continued*)

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SITUATIONS VACANT. Reinforced concrete detailers required by London firm of contractors. Candidates should be capable of building up working drawings from an outline design, and of doing elementary design. BOX 2487, CONCRETE AND CONSTRUCTIONAL ENGINEERING, 14 Dartmouth Street, London, S.W.1.

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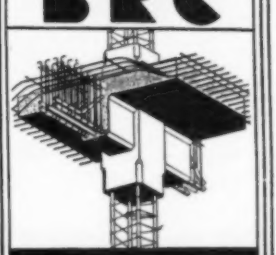

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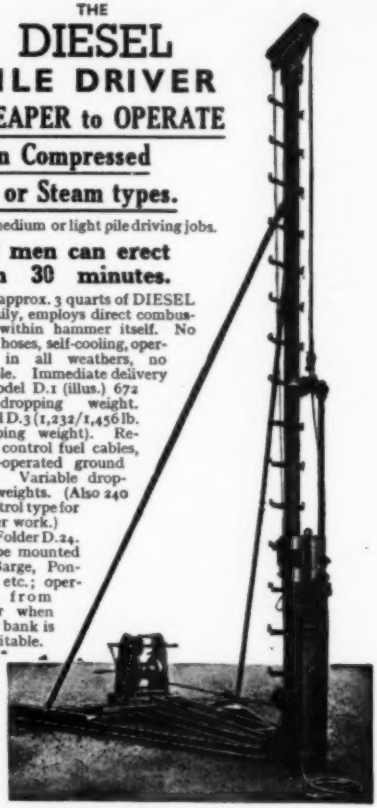

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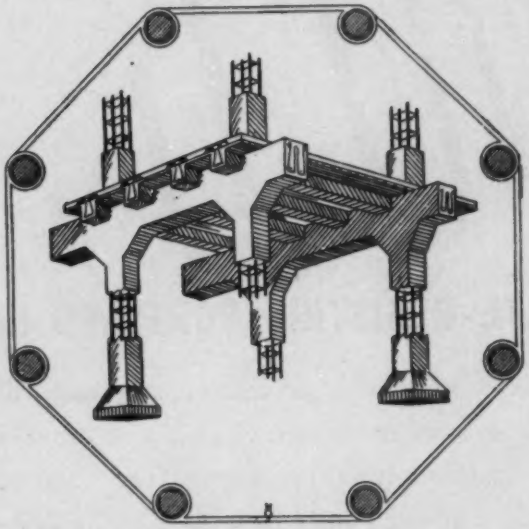

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